Norton Area Hydrotechnical Study
Kennebecasis River

Main Report

The Canada – New Brunswick Flood Damage Reduction Program

FENCO in association with
MACLAREN PLANSEARCH
and GEOCON
Lavalin

March 1984

The cover photograph shows the community of Norton inundated by floodwaters from the Kennebecasis River during the event of February 12, 1981. The photograph was taken by the staff of Environment New Brunswick.

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23 March 1984

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Attention: Mr. J.G. Lockhart, Program Manager
Flood Damage Reduction Program

Norton Area Hydrotechnical Study

Dear Mr. Lockhart:

We take great pleasure in submitting the final version of our report on the Norton Area Hydrotechnical Study, Kennebecasis River. Our final "Supplementary Report on Field Investigations" is also enclosed. The resulting floodlines on the agreed upon interim mapping have been forwarded under separate cover.

It has been a pleasure to work with yourself, your staff and the other members of the Hydrotechnical Subcommittee on this project. We thank you for your assistance and constructive comments throughout its progress. We look forward to being of service to you again in the near future.

Yours truly,

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NORTON AREA HYDROTECHNICAL STUDY

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- Dr. N. Elhadi, Environment New Brunswick
- Mr. B.C. Burrell, Environment New Brunswick
- Ms. E. Langley, Inland Waters Directorate, Environment Canada
- Mr. D. Randall, Inland Waters Directorate, Environment Canada
- Mrs. L. Reid, Norton

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EXECUTIVE SUMMARY

As a means of reducing damages due to flooding, the Province of New Brunswick has instigated a Flood Damage Reduction Program. Through a joint agreement with the Government of Canada, various studies are currently underway or complete to map flood risk areas within the Province. Historical evidence has shown that the Kennebecasis River in the Norton area is prone to flooding on a relatively frequent basis. Hence a study to determine those areas which are subject to flooding on the average once every 20 and 100 years was initiated in November 1982.

The study area comprised of the reach of the Kennebecasis River between Moosehorn Creek and Apohaqui and the first half kilometre of tributary streams within this area. Flooding has been reported on about 20 occasions in this part of the river since about 1854. While flooding of low lying areas occurs almost annually, newsworthy floods, generally those causing property damage or road closures, occur about once every 3 to 5 years. Floods generally occur during the spring freshette but are not restricted to this time of year. Ice jams either within or just downstream of Norton have caused many of these floods. The most recent example of significant flooding occurred in February 1981 when the entire river valley was inundated. At least twelve properties in Norton were surrounded by flood waters and the main road through the town was closed.

This study consisted of the determination of flood profiles using a computerized model known as HEC-2. The first step in the analysis was to calculate design flows with return periods of 1:20 and 1:100 years. This was based upon the 20 years of flow record at the Water Survey of Canada gauge on the Kennebecasis River at Apohaqui and on an analysis of flow records from nearby watersheds. The second phase of the work was to obtain details of the river channel and flood plain through an extensive field survey. This information was used to develop the model of the study area. The model was used to recreate the observed water surface profile for the February 1981 flood and another bankfull flow in order to adjust its parameters to suit this particular river. With this complete, the design flood profiles were computed under ice free conditions. It was concluded that the 1:20 year profile would be very similar to that observed in February 1981. The 1:100 year levels would be about 1 metre above the 1981 levels. It was also concluded that the backwater from the main river would in most cases dominate flood levels on the tributaries examined for the first half kilometre.

The final phase of the analysis was an examination of the potential and historical locations of ice jamming. Based upon historical records and a field reconnaissance four sites were identified. Three of these: at the confluence of Moosehorn Creek, at the old Norton railway bridge and at the confluence of Clements Brook were judged to be significant. Using a special option of the HEC-2 model, the impact of ice jams at these locations on bankfull, 1:20 and 1:100 year flows was examined. It was concluded that water levels would be raised a maximum 1.5 to 2.0 m and that levels would be affected for several kilometres upstream of these jam sites. However, historical evidence and ice scar marks indicated that in practice, ice jams probably do not cause levels to exceed those calculated under 1:100 year open water flow conditions. However, ice jams under bankfull and 1:20 year flow conditions cause a more pronounced effect on the flood profile.

This study recommended that the computed 1:20 and 1:100 year open water flood profiles be used to delineate the corresponding flood risk areas when suitable topographic mapping becomes available. In consideration of flood remedial measures it appears that while some local measures may be effective in Norton, general flood protection for the valley is not feasible. Some improvement with regard to the ice jam situation can probably be achieved by removing the obstruction caused by an S-bend and islands at Moosehorn Creek and by removing the constriction caused by the old railway bridge at Norton. The ice jam situation at the confluence of Clements Brook could also be potentially improved by straightening of the channel at this location. However, there are no flood prone properties immediately upstream of this area and therefore such measures might have lower priority than those at the other ice jam sites. Consideration of the details of these and other flood remedial measures and their cost-effectiveness was beyond the scope of this study.

1.0 INTRODUCTION

1.1 Flood Damage Reduction Program

Flooding in Canada can occur from such diverse sources as spring snowmelt, tropical storms, flash thunderstorms or jamming of ice during spring break-up. Inundation of the floodplain of a river during such times is a natural occurrence. However, the flood damages which often result are not. They are the consequence of man's development on floodplain lands.

Such developments have historically occurred because of the use of rivers as transportation routes, sources of power and water, and because much of the best agricultural land is located within floodplains. The resulting conflict with the river at flood times has led to a variety of approaches to controlling flooding. The earliest records of such attempts in North America to modify the relationship between man and floods stretch back to 1617 to the use of dykes by early French settlers of the Bay of Fundy region to protect areas for agricultural purposes[1]. Methods have generally focussed on various structural measures such as flood control dams, channelization or diversion works. In the absence of such works the alternative has often been the payment of disaster assistance from the public purse to the sufferers of flood damages.

In light of trends towards: increases in flood disaster assistance payments[1], greater pressure for floodplain development because of increasing urban population coupled with escalating land costs, and the potential environmental problems associated with structural flood control measures, it has been recognized that a new and more comprehensive approach to floodplain management is required. Policies based on a full evaluation of both non-structural alternatives, such as restriction of flood vulnerable development in high flood risk areas, as well as traditional structural approaches are necessary. Given this increasing awareness, the federal government undertook in the early 1970's, a comprehensive review of its programs and policies with respect to flood damage. With the benefit of federal-provincial consultations a new National Flood Damage Reduction Program was formulated.

As a pioneer of this new approach, the Province of New Brunswick entered into a "General Agreement Respecting Flood Damage Reduction" with the Government of Canada with the objective of reducing potential flood damage on floodplains and along the shores of lakes, rivers and the sea. This agreement recognizes that the potential for flood damage can be reduced by control of the uses made of areas prone to flooding.

A Flood Risk Mapping Agreement and a Studies Agreement were signed in 1976 and a Flood Forecasting Agreement, to provide an effective flood warning system for the Saint John River Basin, was signed in 1977. All agreements were extended in 1981 for an additional five years. Agreements were also signed to implement remedial works in the Marsh Creek watershed. The City of Saint John was an equal partner with respect to this agreement. The Marsh Creek Agreement is a classical example of the new policy in that both non-structural and structural works are key elements of the basic approach to reduce flood damages.

The Flood Risk Mapping Agreement is the cornerstone of the Flood Damage Reduction Program. Under this Agreement, the flood risk areas along a number of watercourses in New Brunswick are being identified. This information is made available to the general public, to federal, provincial and municipal government agencies and to as many potential investors as possible. Usually, the flood risk map identifies two zones, the floodway and the floodway fringe. In New Brunswick, the floodway is defined as that area flooded on an average of once in 20 years or where there is a 5% chance of flooding in any given year. The floodway fringe makes up the remainder of the flood risk area. The floodway fringe includes all that area flooded on an average of once in 100 years or where there is a 1% chance of flooding in any given year. Once flood risk maps have been prepared for an area, the federal and provincial governments, through a formal designation process, agree not to engage in or fund developments in the floodway and will only participate in funding developments in the floodway fringe provided they are adequately floodproofed. Undertakings for agricultural and recreational purposes are excluded from this policy. Furthermore, once an area has been designated, the governments may withhold disaster assistance payments to developments constructed after designation has been effected.

The Studies Agreement provides for the evaluation of remedial measures to minimize flood damage potential in areas which have a significant flooding problem. Under the agreement, all feasible structural and non-structural measures must be evaluated in terms of effectiveness, cost, benefit, and environmental impact. Structural works include, but are not limited to, flood control dams, dykes, channel improvements and diversions. Non-structural measures include, but are not limited to, floodproofing, zoning, flood warning, emergency measures, land acquisition and watershed management.

One of the areas selected for mapping is the floodplain of the Kennebecasis River and its tributaries in the area from Norton to Apohaqui. This area was chosen because of habitual flooding of residential, commercial and agricultural land in the area. In addition increasing development pressures are being experienced through this area with encroachment onto the floodplain taking place. This report discusses the studies carried out in the preparation of flood risk mapping of this reach of the Kennebecasis River.

1.2 Scope of Study

1.2.1 Study Area

The Kennebecasis River is the last major tributary of the Saint John River before it enters the Bay of Fundy at the City of Saint John. The River's general location is shown on Figure 1.1. The drainage area to the head of Kennebecasis Bay is 1375 km^2. The watershed length is approximately 97 km with its axis in a roughly north-east to south-west direction. As Figure 1.1 indicates, the larger communities located within the watershed are Sussex, Sussex Corner, Apohaqui, Norton and Hampton.

The focus of this study was the floodplain and channel of:

1. The Kennebecasis River from the bridge at Apohaqui to a point 0.5 km downstream of the confluence of Moosehorn Creek with the Kennebecasis River;

2. All tributaries to the above, e.g. Almshouse Brook, to:

    (a) the limit of the backwater effect from the 1:100 year flood, or

    (b) a distance of 0.5 km upstream from their confluence, whichever is greatest.

The approximate limits of the study area are shown in Figure 1.1.

However, because of the nature of the Kennebecasis River, i.e. its shallow slope and the potential effect of the Saint John River, the reach between Kennebecasis Bay and Moosehorn Creek also received attention during this study.

1.2.2 Terms of Reference

This study was carried out in accordance with terms of reference issued by the New Brunswick Flood Risk Mapping Technical Committee[2]. These can be summarized as follows:

1. The Norton Area Hydrotechnical Study has two main components: the development of water surface profiles and a review of the problem of ice jams.

2. The primary purpose of this study is the development of 1:20 and 1:100 year open water flood profiles along the Kennebecasis River between the bridge at Apohaqui and a point 0.5 km downstream of the confluence of Moosehorn Creek with the Kennebecasis River.

3. The Consultant should utilize a mathematical model suitable to the characteristics of the portion of the Kennebecasis River under investigation. Special consideration should be given to defining the downstream boundary conditions of the model.

4. In the final report, the Consultant must present:

   i) a description of past open water flood events;

   ii) the 1:20 and 1:100 year flood profiles for the study portion of the Kennebecasis River (and its tributaries within the study area) in tabular and graphical format;

   iii) a description of the methodology used to develop the flood profiles; and

   iv) a description of all basic data used in the study.

5. On six 1:5000 scale topographical maps with a half metre contour interval (supplied by the client 1)), the Consultant shall:

   i) plot the 1:20 year and 1:100 year flood profiles; and

---

1) Due to fiscal constraints, those maps were not produced during the course of this study and hence the information described was shown on existing less detailed mapping for the interim.

ii) plot the locations of all cross sections used to obtain the water surface profiles.

6. Ice jams have been a major cause of flooding in the Norton Area. For that reason, the Consultant should consider ice jams and associated flood events as part of the hydrotechnical study.

7. The Consultant should present within the final report:

   i) a summary of historic ice jam events;

   ii) a brief discussion of flood damages that have occured in the Norton area due to ice jams;

   iii) a discussion of past ice jam flood levels as it relates to the 1:20 and 1:100 year open water flood profiles;

   iv) a map depicting the locations of past and potential ice jam events;

   v) a discussion of the water surface profiles that may be associated with an ice jam event.

1.2.3 Authorization

On the basis of a proposal submitted in September 1982, Fenco Consultants Ltd., in association with MacLaren Plansearch Inc. and Geocon Inc. were retained by Environment New Brunswick to carry out the Norton Area Hydrotechnical Study. A formal agreement was signed on October 27, 1982.

1.3 Study Approach

The study consisted of the following broadly defined tasks:

- a background information review in which all pertinent records and reports related to hydrology, hydraulics, ice jamming and flood damages were collected and reviewed. Reports of flooding in the area in collected excerpts from newspaper archives were reviewed and discussions were held with local, long standing residents, and with officials of Environment New Brunswick.

- a field reconnaissance and field survey were carried out to obtain cross sections of the floodplain and channel suitable for the establishment of a backwater model.

- monitoring of flood elevations was carried out using five flood crest gauges established within the main study reach.

- hydrologic design flows were computed on the basis of a frequency analysis of flow records available for the Kennebacasis River at Apohaqui (streamflow gauge No. 01AP004) operated by Water Survey of Canada and a sub-regional frequency analysis carried out in connection with the Sussex Area Hydrotechnical Study[3].

- downstream boundary conditions for the study reach were established by using a coarse hydraulic model of the reach of the river between Kennebecasis Bay and the confluence with Moosehorn Creek.

- flood profiles within the study reach were obtained by using a detailed HEC-2 backwater model based upon the field surveyed cross sections and available ortho-photo mapping. The model was calibrated prior to calculation of final profiles based on observations obtained during the flood events of February 12, 1981 and November 13, 1982.

- sensitivity analyses were carried out to determine the impact of variations in flow, Manning's roughness and downstream conditions on levels in the study reach.

- the flood risk areas for the 1:20 and 1:100 year flows were delineated on topographic mapping of the study reach1). The location of the field surveyed sections was also shown on these maps.

- the history of ice jamming in the study area was reviewed and the locations of past jams were mapped.

- the effect of ice jams on flood profiles were examined using the HEC-ICE option of the HEC-2 backwater model.

The following sections of this report discuss these tasks and the results of each activity in greater detail.

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1) The proposed 1:5000 scale mapping was not available due to fiscal constraints, hence the information was presented on a combination of existing maps. These were four 1:10000 orthophoto maps (Nos. 21H/12V-2, 21H/12V-4, 21H/12W-3, 21H/12Z-1) and three 1:2400 scale maps in the Norton area (N.B. Topographic Series Nos. 10Q-31, 10Q-41 and 10Q-42).

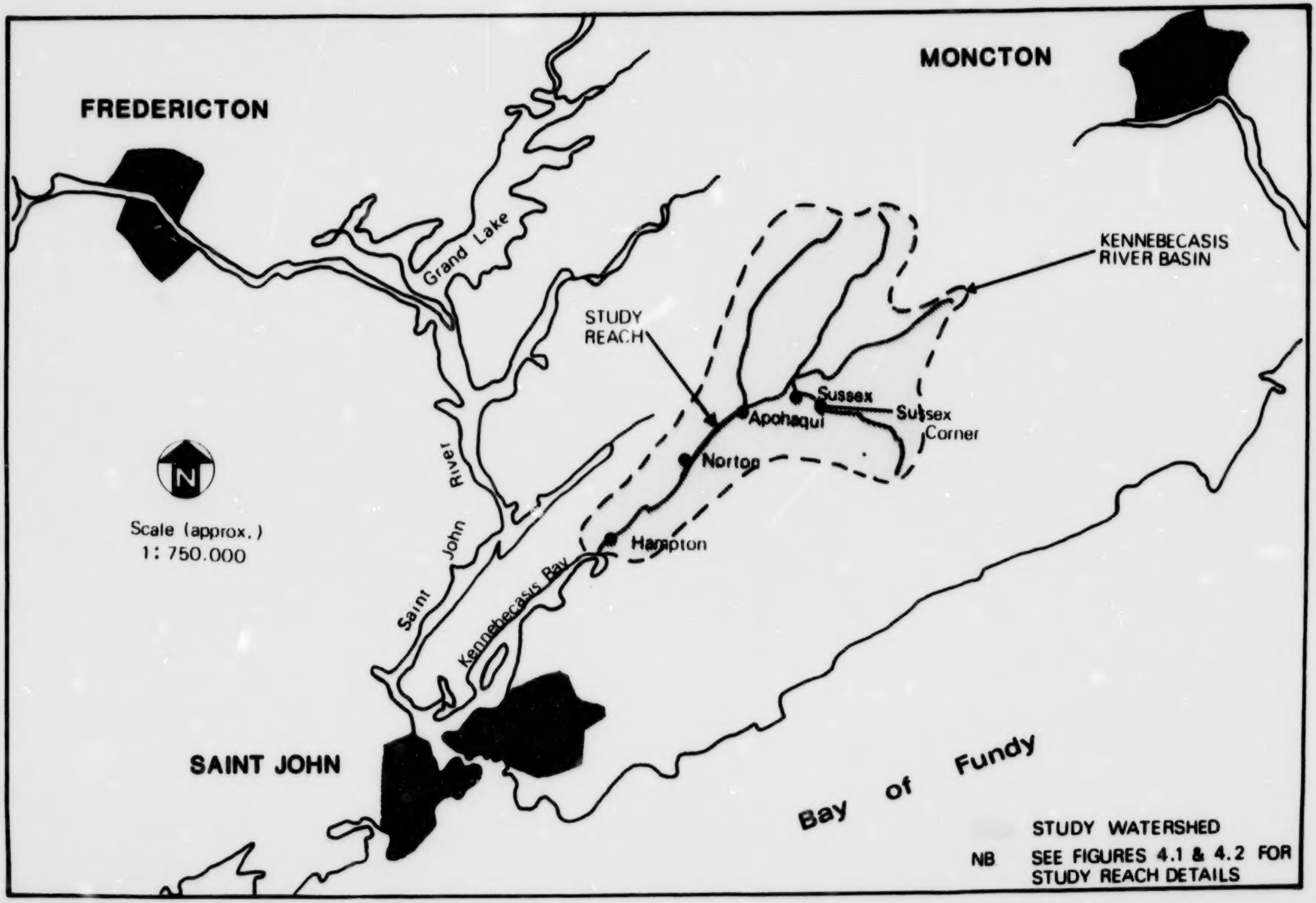
MONCTON
FREDERICTON
Grand Lake
KENNEBECASIS RIVER BASIN
STUDY REACH
Sussex
Sussex Corner
Apohaqui
Norton
Hampton
Saint John River
Kennebecasis Bay
N
Scale (approx.) 1: 750.000
SAINT JOHN
Bay of Fundy
STUDY WATERSHED
NB SEE FIGURES 4.1 & 4.2 FOR STUDY REACH DETAILS

Figure 1.1 General Location of Study Area

2.0 BACKGROUND INFORMATION

2.1 Existing Hydrometeorological Information

2.1.1 Streamflow and Water Level

Streamflow data are collected at the 8 locations shown in Table 2.1 within the general region of the Kennebecasis River excluding points on the Saint John River. Most importantly, a streamflow gauge, No. 01AP004 on the Kennebecasis River, is operated by Water Survey of Canada at Apohaqui at the upper limit of the study area. A continuous record of flows is available from 1962 to date. This data indicates a mean annual flow of about 25 m^3/s at Apohaqui with maximum and minimum daily extremes of 639 m^3/s (recorded on February 12, 1981) and 1.01 m^3/s (recorded on September 14, 1966) respectively. The mean annual flood is approximately 300 m^3/s.

Water level data is available at several points along the Saint John River. Of interest to the current study were water levels in Kennebecasis Bay for the purposes of defining starting conditions for the backwater analysis. The closest points at which such data is available are Indiantown, Station No. 01AP005, and Oak Point, Station No. 1AP003. These stations have 17 and 60 years of record respectively.

2.1.2 Meteorological Data

Galbraith[4] has summarized the meteorological data available within the general region of the Kennebecasis River. As Table 2.2 indicates, there are 8 ordinary climatalogical stations and 3 principal stations within the general

TABLE 2.1

STREAMFLOW DATA AVAILABLE WITHIN THE REGION OF THE KENNEBECASIS RIVER BASIN

River and Gauge Location	Water Survey of Canada Gauge No.	Basin Area (km^2)	Years of Record[1)] Maximum Daily Flow	Years of Record[1)] Instantaneous Peak Flow
Kennebecasis River at Apohaqui	01AP004	1100	21	20
Northwest Oromocto River at Tracy	01AM001	556	20	19
Lepreau River at Lepreau	01AQ001	239	66	19
Magaguadavic River at Elmcroft	01AQ002	1420	54	14
Palmers Creek near Dorchester	01BU004	34.2	16	15
Point Wolfe River at Fundy National Park	01BV006	130	19	19
Upper Salmon River near Alma	01BV007	181	14	12
Ratcliffe Brook below Otter Lake	01BV005	29.3	14	10

1) Up to and including 1982.

TABLE 2.2

CLIMATALOGICAL STATIONS WITHIN THE REGION OF THE KENNEBECASIS RIVER BASIN

Station	Latitude	Longitude	Elevation	Began	Ended
Principal					
Saint John				Jan 1871	Jul 1883
Saint John	45°16'	66°04'	40 m	Jul 1883	Sep 1970
Saint John A	45°18'	66°06'	30 m	Oct 1939	Nov 1951
Saint John A	45°19'	65°53'	109 m	Jan 1952	
Moncton A	46°07'	64°41'	71 m	Dec 1939	
Fredericton UNB	45°57'	66°36'	50 m	Dec 1871	Sep 1952
Fredericton CDA	45°55'	66°37'	40 m	Aug 1913	
Fredericton A	45°52'	66°32'	20 m	Apr 1951	
Ordinary					
Sussex	45°43'	65°31'	21 m	Oct 1897	May 1950
Sussex	45°43'	65°32'	21 m	May 1950	
Penobsquis	45°47'	65°24'	30 m	Nov 1961	May 1970
Penobsquis	45°47'	65°23'	44 m	Jun 1970	Mar 1974
Searsville	45°43'	65°42'	61 m	Dec 1965	
Wolfe Lake	45°40'	65°09'	305 m	Nov 1976	Jul 1979
McManus Hill	45°39'	65°08'		Jun 1935	Aug 1940
Hampton	45°32'	65°50'	16 m	Dec 1881	Mar 1882
Hampton	45°32'	65°50'	16 m	Mar 1884	Apr 1884
Hampton	45°32'	65°50'	16 m	Jan 1934	Jul 1935
Hampton	45°32'	65°50'	16 m	Aug 1936	Dec 1938
Hampton	45°32'	65°50'	16 m	Jul 1950	Jun 1965
Alma	45°36'	64°57'	35 m	Feb 1950	Oct 1965
Alma	45°36'	64°57'	43 m	Oct 1965	
Wiggins Pt.	45°56'	66°00'	8 m	Jun 1970	Oct 1976
Wiggins Pt.	45°56'	66°00'	15 m	Nov 1976	

area1). The station at Sussex has a more or less continuous record from 1897 to date. The remaining stations have periods of record varying from 3 to 103 years in length.

The records at Sussex indicate a normal annual precipitation of 1024 mm of which 835 mm is rainfall. The greatest recorded rainfall for a 24 hour period was 98.8 mm which occurred in April 1962. This corresponded to the greatest recorded flow at Apohaqui prior to the February 12, 1981 flood event. This rainfall compares to an estimated 1:100 year 24 hour rainfall of 116.8 mm[4].

Although the above information is of interest as background to the present study, climatalogical data has not been used directly since flow estimates were derived from the available flow records as described in Section 3.0.

2.2 Historical Flooding

Records of flooding in the Norton area of the Kennebecasis River are available from local newspapers since about 1854. In addition, the recollections of long-standing residents, particularly Mrs. L. Reid, a local historian, were useful in expanding the information obtained from the newspaper reports. From a review of two reports in which relevant articles[5,6] have been assembled, it appears that flooding

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1) Allowing for slight changes in location and name of the station.

of low lying areas is almost an annual occurrence. This will generally occur during the spring freshet but is by no means restricted to this period. Flood reports are also prevalent for the months of January and February, and during the fall. Newsworthy floods, generally those causing damage to property, closure of roads and other transportation routes, occur on average about once every 3 to 5 years in the Norton area. Table 2.3 summarizes the information collected on 20 such floods in the period 1854 to 1982.

The causes of flooding in the study area are diverse as reflected by the different times of year at which flooding occurs. The main causes appear to be:

- snowmelt caused by high temperatures in combination with heavy sustained rainfall

- snowmelt (and possibly rainfall) combined with ice jamming at the confluence of Moosehorn Creek or within Norton itself

- intense rainfall possibly derived from an extra-tropical storm

The most noteworthy recent example of flooding in the Norton area was the event of February 11 and 12, 1981. It was caused by snowmelt combined with heavy rainfall. There is no evidence of ice jamming affecting peak water levels although it is possible that jams occurred earlier in the event. This event is particularly well documented by a set of areal photographs taken by Environment New Brunswick at about the time of the peak of the event. Flooding within Norton was

TABLE 2.3

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
1854 Nov 12-14	Rain	Water rose rapidly from Sussex to Hammond River. Cattle drowned in their stalls.
1887 -	-	No data available although event is mentioned in reports from later years.
1902 Mar 17-23	High temperature and heavy rain. Ice jams.	At Lower Millstream considerable damage occurred when mill dam burst. Highways and bridges in the vicinity of Norton were damaged or swept away. Norton conditions were worst since 1854.
1909 Jan 04-07	Mild weather and heavy rains. Possible ice jams.	14 bridges destroyed in Kings Co. between Sussex and Saint John. At Norton, flooding resulted in the evacuation of several barns. The only method of conveyance was by boat.
1923 Apr 09-12	Snowmelt and ice jams - 3 days of mild weather.	At Apohaqui, cellars flooded. On April 10 the ice broke at Sussex and jammed at Norton. By 9 o'clock of that day, water was flowing over the road. Water rose over road inundating houses and shops, reaching 2 feet (0.6 m) in some cases. Jam broke later that day and water receded quickly. Freshet described as "highest in years."
1923 Apr 29-May 09	Snowmelt, heavy rain, warm temperatures.	At Norton several buildings were threatened but water receded before entering structures. Water to within 6 inches (0.15 m) of B. Perley Wood's drug store floor. At Bloomfield all interval land inundated, 200-300 feet (60-90 m) of road between the bridges submerged. The Kennebecasis River was said to be higher than it was in the memorable floods of 1887 and this was the second flood this spring.

TABLE 2.3 (Cont'd.)

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
1936 Mar 16-25	High temperatures for 2 days. Ice jams.	Flooded areas at Norton, Hampton, Rothesay. Flooding extensive at Norton forcing people to evacuate. Flood waters highest in Norton. Ice over road at Wilcox Brook. Road grade is now 3 feet (0.9 m) higher. Families had to be evacuated from second storey windows.
1945 Feb 05-06	Rain, mild temperatures, ice jams.	At Norton two ice jams brought flood water into several homes and stores near the river. One jam located at the Highway bridge while the other formed about 2 miles (3.2 km) D/S of community. The lower jam was reported to have broken under the pressure before the situation became too serious.
1947 Feb 05	Torrential rains, high temperatures, ice jams.	Ice jams at Norton. Worst flood in at least 6 years. Ice jammed at Norton Bridge and 2 miles (3.2 km) further down river from Norton Bridge. Water in homes and stores.
1950 Nov	Intense rainfall.	Worst fall floods in Kings County since 1938.
1951 Feb 07-08	Mild temperatures, rain, melting snow, ice jams.	At Norton highways were under water at some points and the street leading into Norton covered by more than 1 foot (0.3 m) of water. Other streets in the community were also under water and several homes surrounded. In some instances houses had water on the ground floor. Water approached within 2 feet (0.6 m) of Highway No. 2. Flooding was the result of ice jam 2 miles (3.2 km) downstream.

TABLE 2.3 (Cont'd.)

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
1952 Jan 24-25	Rain, mild temperatures, ice jam.	Recent rains have raised the Kennebecasis to almost flood level several times, but this storm resulted in a heavy ice jam midway between Norton and Bloomfield. At Norton water levels reached "dangerous" levels. Fields were flooded and about "a one-foot (0.3 m) rise would have sent water over the main street in the vicinity of Norton Superior School". Some basement flooding occurred.
1953 Feb 07-08	Torrential rains, mild temperatures, ice jam. 2.70 in (69 mm) of rain at Sussex in less than 24 hours.	In Norton Area an ice jam 2 miles (3.2 km) west of Norton backed up water to the danger point before breaking under its own pressures. Fences were broken and other minor damage was reported to be caused. The water level in the Kennebecasis River was estimated to be 20 feet (6 m) above normal summer levels before the ice jam broke at mid-day February 8th.
1957 Jan 22-23	Mild temperatures, rain and melting snow.	A danger point in the Kennebecasis River, 2 miles (3.2 km) west of Norton, was reported "to be free of ice jams".
1961 May 25-31	Severe extra-tropical storm.	In Kings County, rivers and streams overflowed their banks but no serious damage was reported other than some minor washouts. CPR cancelled trains one being Chipman to Norton. The streams in Kings County were reported to have been higher than the annual spring freshet which was below normal this year.
1962 Mar 31-Apr 04	Heavy rains. 5.04 in (128 mm) at Sussex	At Apohaqui 40,000 board feet (95 m^3) of logs at Jones Bros. Limited sawmill were washed away. At Norton, river inundated stores, homes and cut off the southern section of the community from outside communication. The main street was under 3 feet (0.9 m) of water for several hours.

TABLE 2.3 (Cont'd.)

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
		The Village of Norton suffered the most in terms of flood damage and the storm was described as the worst in the area since the April-May flood of 1923 which was "the greatest freshet since 1887" (Kings County Record May 11, 1923). Two stores, E.K. Cripps and Perkins Hardware were flooded to a depth of 2 feet (0.6 m) or more. The water was reported to have reached the step below the cement entrance to Burley's Mart. Many homes in Norton could be reached only by boat. No ice jam.
1970 Feb 02-06	Heavy rain, high temperatures and ice jams. More than 4 in (200 mm) of rain.	In Kings County heavy damage was reported to have occured to secondary roads and several were closed because of washout. At Norton water was reported to be lapping against some low lying buildings in the village.
1973 May 03	Spring thaw.	N.B. Environment photo shows road at Bloomfield Station under water.
1976 Jan 27-Feb 02	Persistent rain. Mild temperature.	Flooding general in watershed although no direct reports for Norton.
1979 Mar 15	Heavy rain. Snowmelt.	Flooding of low lying areas. N.B. Environment photos show water up to low point on road through Norton and touching back of out-buildings. Also close to road level at Bloomfield Station.

TABLE 2.3 (Cont'd.)

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
1981 Feb 11-13	Rain, mild temperatures, melting snow and ice accompanied by high winds. Some ice jams were also reported but probably not at time of peak flow.	Flooding in low lying areas was reported throughout Kings County. A train bridge abutment at Wilson Brook between Apohaqui and Norton was washed out. At Norton the main road was out and about 12 properties were surrounded by flood waters. The water level at the Apohaqui gauge peaked at a record high. A mile of underground cable was also washed out cutting service to the New Brunswick Telephone customers in the Springfield-Norton area. Extent of flooding well documented by areal photos taken by New Brunswick Environment.

extensive with the main road through the village being closed. At least twelve properties were surrounded by flood waters. The flow at Apohaqui peaked at the highest level on record. Since this event occurred so recently and is well documented, it provided an excellent data set with which to calibrate and check the hydraulic model used in this study.

2.3 Previous Studies

Several earlier studies provided useful background information for the Norton area hydrotechnical study. The following sections briefly describe these studies and indicate the information extracted from them. Further discussion of the results of these studies as they compare with the current study is given where appropriate later in this report. In particular, these studies were:

i) "Hydrotechnical Study, Sussex Area Floodplain"[3]

This study was carried out to obtain 1:20 and 1:100 year open water flood profiles for the Kennebecasis River and its tributaries for the area upstream of Apohaqui through Sussex. The method used was very similar to that adopted in the current study and hence much useful background information was obtained from the earlier study, including:

- flood flow estimates as derived from a sub-regional frequency analysis performed for the Sussex area (see Section 3.1.2 for a detailed description)

- field observations and initial flow estimates for the February 1981 flood

- initial estimates of roughness coefficients for the hydraulic model of the study area.

ii) "Hydrotechnical Studies of the Saint John River, from McKinley Ferry to Lower Jemseg"[7]

This study utilized state-of-the-art unsteady flow modelling techniques to develop 1:20 and 1:100 year open water flood profiles for the lower Saint John River. Hence it provided much useful background information on down stream water levels in Kennebecasis Bay. These were used to determine the appropriate starting conditions for the study area as discussed in Section 4.3 of this report.

iii) "Historical Flood Review of the Upper Kennebecasis River Basin"[5]

"Flooding Events in New Brunswick - an Historical Perspective"[6]

These two studies document the recorded history of flooding in the area as found in local newspapers. Where possible the extent and depth of flooding at specific locales is identified, the extent of damages is documented and the causes of the flooding are presented. As previously discussed, this provided much of the background for Section 2.2 of this report.

iv) "Flood of February 12, 1981 in the Kennebecasis River Basin"[8]

This study was carried out in the aftermath of the February 1981 flood to document the observed flood elevations and flows at the Apohaqui gauge. This was necessary in part because this gauge was washed out by the flood and peak water levels and flow had to be estimated by field observers. A description of the meteorological and antecedent conditions for the event was also provided. The peak flow and water level used in the calibration of the backwater model of the study reach were obtained from this source.

v) "Regional Flood Frequency Analysis of New Brunswick"[9]

This study developed regional flood frequency relationships for New Brunswick as a basis for floodplain mapping studies in identified flood prone areas. The flows developed for the Kennebecasis River were superceded by those of the Sussex area hydrotechnical study.

vi) "Climatological Data for Sussex, N.B. Area in Relation to Floodplain Studies"[4]

This study provides a compilation of available climatic data in the region of the Kennebecasis River. Temperature and precipitation summaries are included. Rainfall intensity-frequency estimates and probable maximum precipitation estimates are included. Discussions of storm transposition and the applicability of the data to the Kennebecasis River are also given. Some of this information has been discussed in Section 2.1 although it has not been directly utilized in the current study.

3.0 HYDROLOGIC ANALYSIS

As indicated in the Terms of Reference for the Norton area hydrotechnical study, flood profiles were required for the 1:20 and 1:100 year flows for open water conditions. In addition, flows were required for calibration of the backwater model and as a basis for examining the impact of ice jams on water levels in the study area. The following sections discuss the derivation of these flows from the streamflow records available within the area.

3.1 Open Water Flood Flow Estimates

Flood flow estimates can be derived by various techniques including:

- single station frequency analysis of flow records at the site of interest
- regional frequency analysis of flow records within the general area of interest
- computer simulation using a mathematical watershed model.

The choice of method in a particular situation is governed by the availability and length of streamflow record at the point of interest. Since flood flow estimates are often required for relatively rare frequencies of occurrence based upon limited data, it is often advisable to derive them by several methods. A choice can then be made from amongst the resulting estimates.

For this study flood flow estimates were required from Kennebecasis Bay to Apohaqui on the main river and on a number of small tributaries between Norton and Apohaqui. Since flow records are available at only one point, i.e. the gauge at Apohaqui, and this has a relatively short period of record, a method other than single station frequency analysis was required. As discussed in Section 2.3, several earlier studies[3,8,9] have provided hydrologic information of direct relevance to this analysis. As the following sections indicate, the flows required for this study were derived from these earlier studies.

3.1.1 Regional Flood Frequency Estimates

A comprehensive regional flood frequency analysis for the Province of New Brunswick was completed under the auspices of the Flood Damage Reduction Program in 1977[9]. The province was divided into three hydrologically homogeneous regions: the North Region, the South Region and the main Saint John River Region. The current study area is within the South Region but in an area noted as a transition zone. The regional analysis provided flow estimates at 3 points within the study area on the main branch of the Kennebecasis River.

The flood flow estimates at Apohaqui, the site of the streamflow gauge No. 01AP004, were derived by a technique known as the Modified Index Method. In this technique the mean annual flood is evaluated from the station records and a dimensionless index curve is applied to calculate 1:20 and 1:100 year flood peaks. The dimensionless index curve gives the ratio of floods of higher return period to the mean annual flood and was derived by regionalizing the curves from individual stations[9]. This technique was found to give superior flood flow estimates if at least 7 years of flow record were available. Table 3.1 shows the resulting estimates at Apohaqui.

For the estimates at Norton and Moosehorn Creek, the above technique was not applied directly since no flow records are available. However, these points are on the same river channel as Apohaqui and differ in drainage area by less than 25% from that at Apohaqui. Hence, the flows were obtained by adjusting the Apohaqui flows by the procedure presented in Section 5.1.3 of the Regional Flood Frequency Analysis Report[9]. The resulting flows are shown in Table 3.1.

3.1.2 Flood Flow Estimates - Sussex Area Hydrotechnical Study

As a basis for the development of 1:20 and 1:100 year flood profiles in the Sussex area of the Kennebecasis River, revised estimates of the 1:20 and 1:100 year flow were developed[3]. This was felt to be required because of the availability of about 4 additional years of flow data since completion of the provincial regional frequency analysis [9], and because of the position of this area at the transition between the North and South flood frequency regions. A subregional frequency analysis was carried out using streamflow records from the 8 stations shown in Table 2.1. The following relationships were developed for estimating peak instantaneous flows:

TABLE 3.1

COMPARISON OF FLOW ESTIMATES FOR THE KENNEBECASIS RIVER BY VARIOUS METHODS

			Return Period			
			20		100	
Location	Drainage Area (km^2)	Method	Flow (m^3/s)	Confidence Limits (%)	Flow (m^3/s)	Confidence Limits (%)
Apohaqui	1100	R.F.F.A.1)	600	+61, -38	880	+71, -41
		S.F.F.A.2)	630	+80, -44	866	+87, -46
		Station Frequency to 1979	624	Unknown	812	Unknown
		Station3) Frequency to 1982	696	+46, -31	993	+70, -40
Norton	1150	R.F.F.A.	620	+61, -38	910	+71, -41
		S.F.F.A.4)	649	+80, -44	890	+87, -46
		Selected Design Flows	717	+46, -31	1020	+70, -40
Downstream of Moosehorn Creek	1260	R.F.F.A.	670	+61, -38	980	+71, -41
		S.F.F.A.4)	688	+80, -44	942	+54, -46
		Selected Design Flows	760	+46, -31	1080	+70, -40

1) Regional Flood Frequency Analysis for New Brunswick[9].

2) As presented in Sussex Area Flood Frequency Analysis[3].

3) Selected design flow for this location.

4) Based on equation presented in Sussex Area Flood Frequency Analysis[3].

$$Q_{20} = 6.74\ A^{0.648}$$
$$Q_{100} = 10.80\ A^{0.626}$$

where Q_{20} is the 1:20 year peak instantaneous flow, m^3/s

Q_{100} is the 1:100 year peak instantaneous flow, m^3/s

A is the drainage area, km^2

The only parameter found to be significant was the drainage area at the point of interest. Areas were derived from 1:50000 scale topographic mapping upon which the watershed boundaries had been delineated. The resulting flow estimates at Apohaqui, Norton and at the downstream limit of the study area are shown in Table 3.1.

3.1.3 Single Station Analysis

Since 3 years of additional flow data were available since completion of the Sussex area hydrologic analysis, including a new flow of record on February 12, 1981, an updated frequency analysis of the flow record at Apohaqui was carried out. Annual maximum instantaneous peak flows for the period 1962 to 1982[1] were analysed using the Flood Damage Reduction Program Flood Frequency Analysis (FDRPFFA) computer program[10]. Figure 3.1 shows the resulting flow frequency curve for the three parameter Log-normal distribution. This distribution was chosen for compatibility with the previous studies[3,9]. Table 3.1 shows the resulting estimates of the 1:20 and 1:100 year peak flow at Apohaqui. Also indicated are the estimates of the same flows based upon the data to 1979 as presented in the Sussex study[3]. The inclu sion of the new flow of record has increased the resulting 1:20 year and 1:100 year estimates by 12 and 22 percent respectively.

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[1] The peak flow for the year 1978 is not available.

3.1.4 Design Open Water Flows Based on Single Station Analysis

Considering the estimates at Apohaqui, the only location at which flow records are available, the highest values are given by the updated single station frequency analysis. This reflects the inclusion of the new flow of record of February 12, 1981. It should be noted, however, that this flow is not a high outlier in the annual maximum series being only 5 and 13 percent higher than the second and third highest flows on record. The record at this gauge has a duration of 20 years and, as shown by Table 3.1, the 95 percent confidence limits respectively for the flow estimates are narrower than for the alternative methods. Hence the updated single station frequency analysis values were selected as design flows for the Apohaqui location.

Since flow records are not available at other locations in the study area the relationships of the subregional frequency analysis for the Sussex area were adopted to derive the remaining required flows. However, to ensure compatibility with the flows at Apohaqui, the flows derived by this approach were adjusted. The ratio of the flow at Apohaqui given by the updated frequency analysis to that given by the subregional analysis was used, i.e. factors of 1.10 and 1.15 for the 1:20 and 1:100 year return periods respectively. In effect this increases the constant multiplier in the relationships given in Section 3.1.2 but retains the same relationship to drainage area.

The final design flows for the 1:20 and 1:100 year return periods at various points on the Kennebecasis River and on the tributaries within the study reach are given in Table 3.2.

3.2 Winter Flood Flow Estimates

Flow estimates were required for the purpose of examining the impact of ice jams at locations identified as historical or potential jam sites. As described in Chapter 6.0, this was carried out by use of the HEC-ICE option of the HEC-2 backwater model. Ice jams have occurred throughout the winter period during which the majority of the peak annual flows have also occurred. Hence the flow estimates made for open water conditions, Section 3.1.4, were also utilized in the ice jam investigation. The impact was investigated using the range of flows from bankfull (as discussed in Section 3.3) to the 1:100 year return period flow as shown in Table 3.2.

3.3 Flows for Calibration Events

In order to verify the performance of the backwater model of the study area, observed flows and water levels were utilized to calibrate and check the model. Since observed flows were only available at the Apohaqui flow gauge, flows at the other points were estimated from the flow at Apohaqui. The method utilized was similar to that used in deriving the design flows. The observed flow at Apohaqui was adjusted using the

TABLE 3.2

FINAL DESIGN FLOWS FOR THE 1:20 AND 1:100 YEAR RETURN PERIOD FOR THE KENNEBECASIS RIVER AND TRIBUTARIES

Watershed	Location	Section No.	Drainage Area (km^2)	Design Flows (m^3/s) 1:20 Year	Design Flows (m^3/s) 1:100 Year
Kennebecasis River	Oliver Pt.	46.0	1963	1013	1426
	Hampton	40.0	1376	804	1142
	Bloomfield	36.4	1269	763	1086
	Moosehorn Creek	31.0	1253	758	1077
	Almshouse Brook	27.0	1182	729	1039
	Halfway Brook	18.0	1136	711	1012
	Apohaqui	1.0	1100	696	993
Clements Brook	Mouth		12.6	39	61
Moosehorn Creek	Mouth		67.3	114	173
Almshouse Brook	Mouth		32.1	71	109
Halfway Brook	Mouth		6.2	24	39

drainage area factor derived in the Sussex area subregional frequency analysis, namely:

$$Q_C = Q_A \left(\frac{A_C}{A_A}\right)^{0.648}$$

where Q_C is the flow at the calibration point, m^3/s
Q_A is the observed flow at Apohaqui, m^2/s
A_C is the drainage area at the calibration point, km^2
A_A is the drainage area at Apohaqui, i.e. 1100 km^2

This method was used to obtain flow estimates for the two selected calibration events, i.e. the flow of record of February 12, 1981 and a bankfull flow observed on November 13, 1982. The selection of these events will be discussed further in Section 4.4. Flows were developed for only the main stem of the Kennebecasis River since no water level data were available for the tributaries within the study area with which to calibrate their models. This was not of major concern, however, since preliminary computations had indicated that the backwater from the main stream would dominate levels on these tributaries for the design flows.

The calibration flows derived for these two events are presented in Table 3.3.

TABLE 3.3

ESTIMATED FLOWS FOR SELECTED CALIBRATION EVENTS FOR THE KENNEBECASIS RIVER

Location	Section No.	Calibration Flow	
		November 13, 1982	February 12, 1981
Oliver Point	46.0	135.1	930
Hampton	40.0	107.2	738
Bloomfield	36.4	101.8	701
Moosehorn Creek	31.0	101.0	696
Almshouse Brook	27.0	97.2	669
Halfway Brook	18.0	94.9	653
Apohaqui[1]	1.0	92.8	639

1) Flow as measured at W.S.C. gauge

3.4 Summary of Hydrologic Analysis

Flood flow estimates were developed for the 1:20 and 1:100 year return periods for various points on the main branch of the Kennebecasis River between Kennebecasis Bay and Apohaqui and on the tributaries within the study area. These were obtained from an updated frequency analysis of 20 years of flow record at Apohaqui. For points downstream of Apohaqui the subregional frequency analysis carried out for the Sussex area hydrotechnical study was applied with an adjustment factor to ensure agreement at Apohaqui. The resulting design flows are shown in Table 3.2. These flows were also used as the basis for examining the impact of ice jams at known locations of jamming. A further set of flows were developed for the main stem of the Kennebecasis River for two specific events, i.e. February 12, 1981 and November 13, 1982. These were used for calibration of the backwater model.

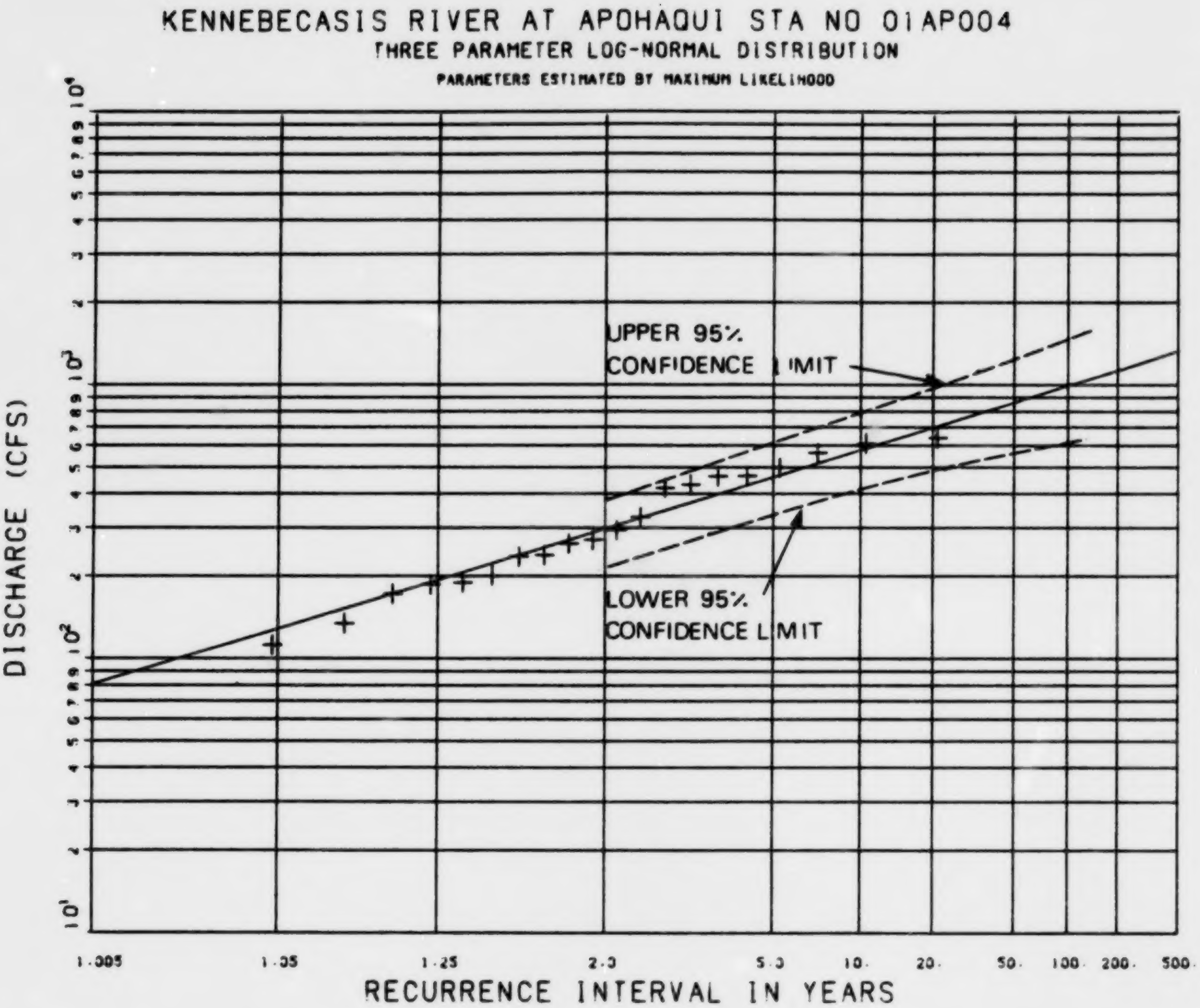

Figure 3.1: Instantaneous Peak Flow Frequency Analysis for the Kennebecasis River at Apohaqui, 1962 - 1982 (excluding 1978)

4.0 HYDRAULIC ANALYSIS

4.1 Introduction

For a given flow in a river it is possible to calculate the corresponding water levels knowing certain physical characteristics of the channel and flood plain such as the cross-section, the roughness and the slope. The technique is known as backwater analysis since downstream conditions control upstream water levels under normal flow conditions. For natural channels and valleys of complex shape it is a very tedious and time consuming process to carry out the necessary calculations by hand. Hence computerized models are used. While the flow-water level relationship is in fact dynamic it can often be approximated by a steady-state condition using the equations of gradually varied flow. One widely used model based on this principle is HEC-2[11] developed by the U.S. Army Corps of Engineers.

The HEC-2 model was selected for use in the Norton area hydrotechnical study. A detailed model of the study area was prepared on the basis of the field surveyed topographic information and available orthorphoto mapping. Water levels on the Saint John River form the downstream boundary conditions for the Kennebecasis River. Since the impact of these levels on the study reach was unknown, a coarse HEC-2 model of the reach from Kennebecasis Bay to the downstream limit of the study area was developed. This was used to examine the sensitivity of starting water levels for the study area to variations on the Saint John River. Having completed this analysis, the main model was calibrated using observed water levels from two recent high flow periods. The calibrated model was then used to determine flood water level profiles for the 1:20 and 1:100 year design flows. Finally, the effect of varying the parameters of the HEC-2 model of the study area and of varying the estimated flows was examined. The following sections discuss this analysis.

4.2 Development of the HEC-2 Backwater Model

4.2.1 Field Surveys

The main requirement for development of a HEC-2 backwater model of a river reach is the availability of cross-sections of the river and its floodplain. These can be obtained from detailed topographic mapping or from field surveyed data. In the absence of detailed mapping of the study area the latter approach was selected for this study. Full details of the field survey are presented in the "Supplementary Report on Field Investigations" under separate cover. In summary the following field studies were carried out:

i) Vertical control points referenced to Geodetic Survey of Canada (G.S.C.) datum were established along the CNR track on the south side of the Kennebecasis River at the beginning of each cross-section.

ii) A total of 34 cross-sections was surveyed within the study area using an electronic distance measuring device and theodolite. As shown on Figure 4.1, twenty-nine of these sections were on the main stream between Moosehorn Creek and Apohaqui1).

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1) The additional 5 sections (Nos. 25.1, 25.2, 25.3, 25.4 and 1) were obtained from earlier surveys in the Norton area.

The locations were selected at about 500-600 m intervals or at major changes in channel or flood-plain characteristics. Three sections (Nos. 33, 34 and 35) were located just downstream of the main study reach (see Figure 4.2) and were obtained to provide accurate starting conditions for the model. The remaining two sections were located on Moosehorn Creek. Plots of these sections are included in Appendices A and B of the supplementary field report.

iii) A total of 12 cross-sections of the river channel within the study reach were determined by sounding from a boat. These were chosen to match the locations of 11 of the flood plain cross-sections (Nos. 1, 3, 7, 10, 13, 17, 20, 23, 28, 31, 33). It was not necessary to obtain river channel sections at each flood plain section because of the uniformity of the channel. However, an additional ten centre channel soundings were obtained to ensure accuracy in interpolating the channel invert for the intermediate sections (Nos. 8, 12, 14, 15, 18, 22, 26, 29, 30 and 32).

iv) Channel cross-sections were obtained in the area downstream of the main study reach at the Bloomfield and Hampton bridges. The river bottom was sounded from the bridge deck and referenced to Geodetic datum. These sections were used in combination with information from 1:10000 orthophoto mapping to prepare the coarse backwater model of the downstream reach. The locations of the modelled sections are shown on Figure 4.2.

v) Details of all structures crossing both the main Kennebecasis River and its tributaries within the study area were obtained. These details included width and height of opening, elevation of deck, obvert of the bridge deck and size and arrangement of abutments and piers. Sketches of these bridges with their dimensions are presented in section 4.0 of the supplementary report. Additional data was obtained from construction drawings supplied by the New Brunswick Department of Transportation.

vi) From a review of historical information regarding flooding in the Norton area various high water marks for events in 1939, 1962 and 1981 were identified. Where feasible these levels were identified in the field and referenced to Geodetic datum. In addition any scars on streamside vegetation or structures thought to have been caused by ice action were identified and surveyed into the datum. A summary of this information is shown in Figures 8-1 and 8-2 and in Tables 2 and 3 of the supplementary report.

vii) In order to obtain water level information for calibration purposes, a series of 5 crest gauges were established within or just below the study area. These were located at Bloomfield bridge, at Norton, at the confluence of Secord Brook, on Almhouse Brook at Highway No. 121 and on Moosehorn Creek at Highway No. 2. These gauges were monitored from late November 1982 to April 26, 1983. During this period no major events occurred. The spring freshette was far below normal because of the very light snowfall of the winter of 1982/83. However, prior to the installation of the crest gauges a flow sufficient to raise water levels to approximately a bankfull condition was observed on November 13, 1982. Field staff were able to monitor this event and record high water marks at the same locations at which crest gauges were installed. This information provided a basis for a suitable calibration event for the HEC-2 model. Details of the measurements are given in Table 5 of the supplementary report.

viii) As part of the studies of ice jam effects on water levels in the study area, a field reconnaissance was conducted on March 30, 1983. This included visits to historical and potential ice jam sites and discussions with local residents and officials of Environment New Brunswick involved in ice observations. Since there was relatively little ice because of the mild winter of 1982/83 it was not possible to observe ice jamming during spring break-up.

4.2.2 Channel and Flood Plain Characteristics

Figure 4.2 shows the entire reach of the Kennebecasis River from Kennebecasis Bay to Apohaqui. Within the study area itself the river channel is essentially quite uniform with a width of about 50 to 60 m and depth to top of bank of 2 to 3 m. The channel is relatively straight with a few more abrupt bends such as at the confluences with Moosehorn Creek and Secord Brook. Under normal conditions the water is about 1 m deep with occasional shallow sections at small rapids and sand/gravel bars. The immediate river banks are generally lined with long grasses or reeds and small brush or trees. Beyond this fringe the land is generally agricultural on the gently sloping floodplain. The floodplain varies in width from about 200 m to 750 m. In the lower reaches the width is about 500 m narrowing to about 200 m near Apohaqui. The floodplain area is bounded by Highway No. 121 on the north and by the CNR tracks and Highway No. 2 on the south, all of which run generally parallel to the river. The river is crossed at Hampton, Bloomfield, Norton and Apohaqui by roads which are carried on embankments above the general level of the floodplain.

Downstream of the main study area the river begins to meander more extensively within the floodplain. The floodplain widens to as much as 2.5 km in the Hampton area and contains extensive areas of marshland.

4.2.3 Model Structure

The HEC-2 model developed for this study consists of three main parts:

- a detailed model from downstream of Moosehorn Creek to Apohaqui. As shown in Figure 4.1 this model contains 34 cross-sections of the river and floodplain1). Figures 4.3 and 4.4 show typical HEC-2 cross-sections for a river/flood plain section and at a bridge struc-

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1) These consist of 29 field surveyed sections, 4 from previous surveys in connection with the Norton bridge reconstruction and 1 from the Sussex area study[3].

ture. As previously described, these were obtained from field survey data, 1:10000 orthophoto mapping and bridge drawings from the New Brunswick Department of Transportation. The orthophoto mapping was used to extend the cross-sections to a common maximum elevation of 20 m G.S.C. in locations where the field survey did not reach this elevation. This was required to provide a common scale for computer plotting of the sections.

- a coarse model of the reach from Oliver Point on Kennebecasis Bay to the lower limit of the study area. As shown in Figure 4.2, this model contains 20 cross-sections spaced approximately 1 km apart. These were derived from 1:10000 Orthophoto mapping, field cross-sections at Bloomfield and Hampton and bridge drawings from the New Brunswick Department of Transportation. The three most upstream sections (Nos. 33, 34 and 35) were obtained from the field survey.

- four tributary models covering the lower 0.5 km of Moosehorn Creek, Almhouse Brook, Halfway Brook and Clements Brook. These models were based on field cross-sections, bridge details obtained in the field and 1:10000 orthophoto mapping. Section locations are shown on Table 5.2.

4.3 Sensitivity to Downstream Water Levels

Since downstream conditions will, under normal circumstances, control water levels in a river it is important in backwater analysis to determine the 'correct' starting water level at the downstream boundary. The Kennebecasis River is a tributary of the Saint John River, a major river on which fluctuations of up to 5 metres are possible. It was therefore important to determine to what extent these fluctuations would affect water levels within the study area. While this reach is almost 50 km from the Saint John River, the slope in the lower reaches of the Kennebecasis River is very slight. Hence high water on the Saint John River would be expected to affect a considerable length of this lower reach. Should this effect extend undiminished well into the study area then further consideration would be necessary both to the choice of modelling technique and to the appropriate choice of downstream condition taking into account the timing of peak flows on both rivers. Should the effect be essentially damped by the beginning of the study area then the choice of level on the Saint John River would not be critical.

In order to examine this sensitivity, the coarse HEC-2 model of the lower reach of the river, described in Section 4.2.3 and shown on Figure 4.2 was utilized. A series of simulations were carried out for the 1:20 and 1:100 year design flows for a wide range of water levels on the Saint John River. The range chosen was from 1.0 m GSC, the minimum water level recorded at Oak Point on the Saint John River, to 5.8 m GSC. The latter is the upper 95 percent confidence limit of the 1:100 year water level at the same location as determined in a recent study of this river[7]. Since Kennebecasis Bay which joins the Saint John River to the Kennebecasis River is approximately 1 km wide and 30-40 m deep it was assumed that this body of water would act as a storage reservoir with a level surface. Hence the level on the Saint John River was transferred directly to the start of the Kennebecasis River at Oliver Point.

The sensitivity analysis was initially carried out prior to development of the HEC-2 model of the main study area in order to confirm that the choice of technique was correct. As the results indicate, water levels in the study area are relatively unaffected by the wide range of downstream levels tested. Hence the chosen method was confirmed as being suitable. Upon completion of the calibration of the HEC-2 model which included the downstream reach, see Section 4.4, these simulations were repeated with the final model. The results presented in Table 4.1 are from these final simulations.

As anticipated, the water levels in the Kennebecasis River in the downstream reaches are quite sensitive to the water level on the Saint John River. At Hampton, for example, the 4.8 m range in starting levels has diminished to only 1.91 m and 1.48 m for the 1:20 and 1:100 year flows respectively. However, by Bloomfield the effect is damped to 0.7 m and 0.49 m for these same flows. At the start of the study area the range is only 0.5 m and 0.27 m. It is noticeable that for a wide range of starting levels, i.e. from low water to the 1:20 year level, the range is only 0.04 m at this location. Given the difference in sizes of the Saint John River and Kennebecasis River watersheds, it appears extremely unlikely that downstream levels outside this range would coincide with peak water levels on the Kennebecasis River. For example, the level at Oak Point was 1.56 m for the flow of record at Apohaqui on February 12, 1981. The final choice from within this range was therefore not critical and hence the mean water level of 1.55 m GSC for the 60 year period of record at Oak Point was selected as the downstream condition. This gives starting water levels of 7.24 m and 8.09 m at the downstream boundary of the main study reach for the 1:20 and 1:100 year design flows. By comparison, the estimated water

TABLE 4.1

RESULTS OF SENSITIVITY ANALYSIS OF DOWNSTREAM WATER LEVELS

Location	Return Period of Flow	Estimated Water Level (m) Starting Level (1.0 m)[1)]	 Starting Level (4.4 m)[2)]	 Starting Level (5.8 m)[3)]
Oliver Point (Section 46)	20	1.00	4.41	5.84
	100	1.00	4.41	5.84
Hampton (Section 40.1)	20	4.12	4.90	6.03
	100	4.72	5.26	6.20
Bloomfield (Section 36.1)	20	5.95	6.12	6.65
	100	6.68	6.80	7.17
Moosehorn Creek (Section 32)	20	7.24	7.28	7.74
	100	8.08	8.12	8.35
Norton (Section 25.2)	20	8.79	8.80	8.94
	100	9.69	9.71	9.78
Apohaqui (Section 2)	20	13.89	13.89	13.90
	100	15.11	15.11	15.11

1) Low water level measured at Oak Point.

2) 1 in 20 year water level at Oak Point[7].

3) 1 in 100 year upper 95% confidence limit at Oak Point[7].

level at this location for the February 12, 1981 event was 7.34 m. Although there are no records available at this point on the river, it appears that the 1:20 year level agrees quite closely with the level for the February 12, 1981 event which was estimated to have a return period of between 1:15 and 1:20 years.

For the remainder of the backwater analysis, the complete HEC-2 model from Kennebecasis Bay to Apohaqui was utilized, ie. the coarse downstream model was combined with the more detailed model of the actual study reach.

4.4 Calibration and Verification of HEC-2 Model

The HEC-2 model is based upon the solution of the equations of gradually varying flow which include Manning's equation as an estimate of the friction slope between two cross-sections. Manning's roughness coefficient is an empirical parameter which must be estimated for a specific location. While guidelines are available for the choice of suitable values knowing the characteristics of the channel and flood plain (see references [12] and [13]), the most accurate method is to calibrate the model using known flows and observed water levels. The process of calibration involves selecting trial values of the calibration parameter, simulating the flood profile and comparing to the observed water levels. The process is repeated until satisfactory agreement is achieved.

For the study area, suitable data was available for calibration purposes for two recent high flow events, November 13, 1982 and February 12, 1981. While high water mark data was obtained for other events it was not extensive enough to be useful for calibration purposes. In addition, the two events for which data was available were ideal in that:

- the November 13, 1982 event produced water levels at or just below bankfull elevations providing data for an independent calibration of the roughness coefficient of the channel without the need to consider the floodplain roughness. The frequency analysis indicated a return period of about 1 year for this event.

- the February 12, 1981 event produced the highest flow on record at the Apohaqui gauge. The frequency analysis indicated a return period of about 1:15 to 1:17 years. Large areas of the floodplain were inundated and this was well documented by a series of air photographs taken by New Brunswick Environment at or close to the time of peak flow. High water marks identified from these photographs were surveyed into datum during the field survey. These provided excellent data for calibrating the roughness coefficient of the overbank/floodplain areas.

High water mark data for these two events is presented in Table 4.2. The results of the calibration of the HEC-2 model are also shown in Table 4.2 and the profiles are compared in Figures 4.5 and 4.6. In general the accuracy of the simulated profile is very good, particularly within the study reach. Maximum deviations from observed levels are +0.17 m and -0.09 m for the November 13, 1982 event and +0.40 m and -0.37 m for the February 12, 1981 event. It should be noted, however, that the maximum deviations occur in the area downstream of the main study reach. Within the study area, the differences vary from 0.01 m to 0.17 m. For both events, the

TABLE 4.2

COMPARISON OF OBSERVED AND SIMULATED WATER LEVELS FOR CALIBRATION EVENTS

		November 13, 1982			February 12, 1981		
Location	Section No.	Observed Level (m)	Simulated Level (m)	Difference (m)	Observed Level (m)	Simulated Level (m)	Difference (m)
Hampton	40.3	-	-	-	4.40	4.03	-0.37
Bloomfield	36.3	3.00	2.95	-0.05	5.40	5.80	+0.40
Norton	25.3	5.20	5.30	+0.10	8.65	8.66	+0.01
	13.0	7.50	7.67	+0.17	-	-	-
Apohaqui	1.0	9.80	9.71	-0.09	13.80	13.63	-0.17

N.B. Starting water levels at Kennebecasis Bay were 1.62 m and 1.56 m for the November 13, 1982 and February 12, 1981 events, respectively, based on Water Survey of Canada gauge, 1AP003.

absolute average error between observed and simulated water levels is approximately 4% of the observed depth.

The calibrated values of Manning's roughness coefficient are shown in Table 4.3. The values for the channel are typical of recommended values for the type of channel within the modelled area, i.e. relatively uniform, wide and smooth bottomed. A comparison with values derived in the Sussex Area study[3] for the Kennebecasis River upstream of Apohaqui indicates the same channel roughness value. Overbank roughness values are relatively high for the type of vegetation found on the Kennebecasis River floodplain. However, it appears that this indicates that not all of the floodplain which is inundated is acting as conveyance for the flow. In other words, there is overbank storage in some areas in which the model simulates flow conveyance. Hence to compensate and reduce the total flow on the overbank a high Manning's n is required. In addition, the "effective" roughness may be somewhat higher than it would appear from field inspection because of the relatively low depth of water in some areas of the floodplain. Floodplain roughness was also found to be relatively high in the Sussex area of the Kennebecasis River[3]. It should also be noted that the sensitivity analysis of water level versus Manning's n, described in Section 5.2, indicated that levels were not greatly affected by the choice of overbank roughness. This is because the majority of the flow (about 85%) is carried within the channel rather than the floodplain. This is clearly shown by the effective Manning's n for the section (based on a flow weighted average) which is very close to the channel value (see Table 4.3).

TABLE 4.3

CALIBRATED VALUES OF MANNING'S ROUGHNESS COEFFICIENT

Location	Section No.	Manning's Roughness Coefficient Channel	 Overbanks	Composite[2)]
Kennebecasis Bay to Hampton	46.0 to 40.3	0.03	0.20	0.05
Hampton to Bloomfield	40.4 to 36.3	0.03	0.10	0.04
Bloomfield to Apohaqui	36.4 to 1.0	0.03	0.175	0.05
Upstream of Apohaqui[1)]	-	0.03	0.10	-

1) From Sussex Area Hydrotechnical Study[3].

2) The composite value is a flow weighted average of the channel and overbank values.

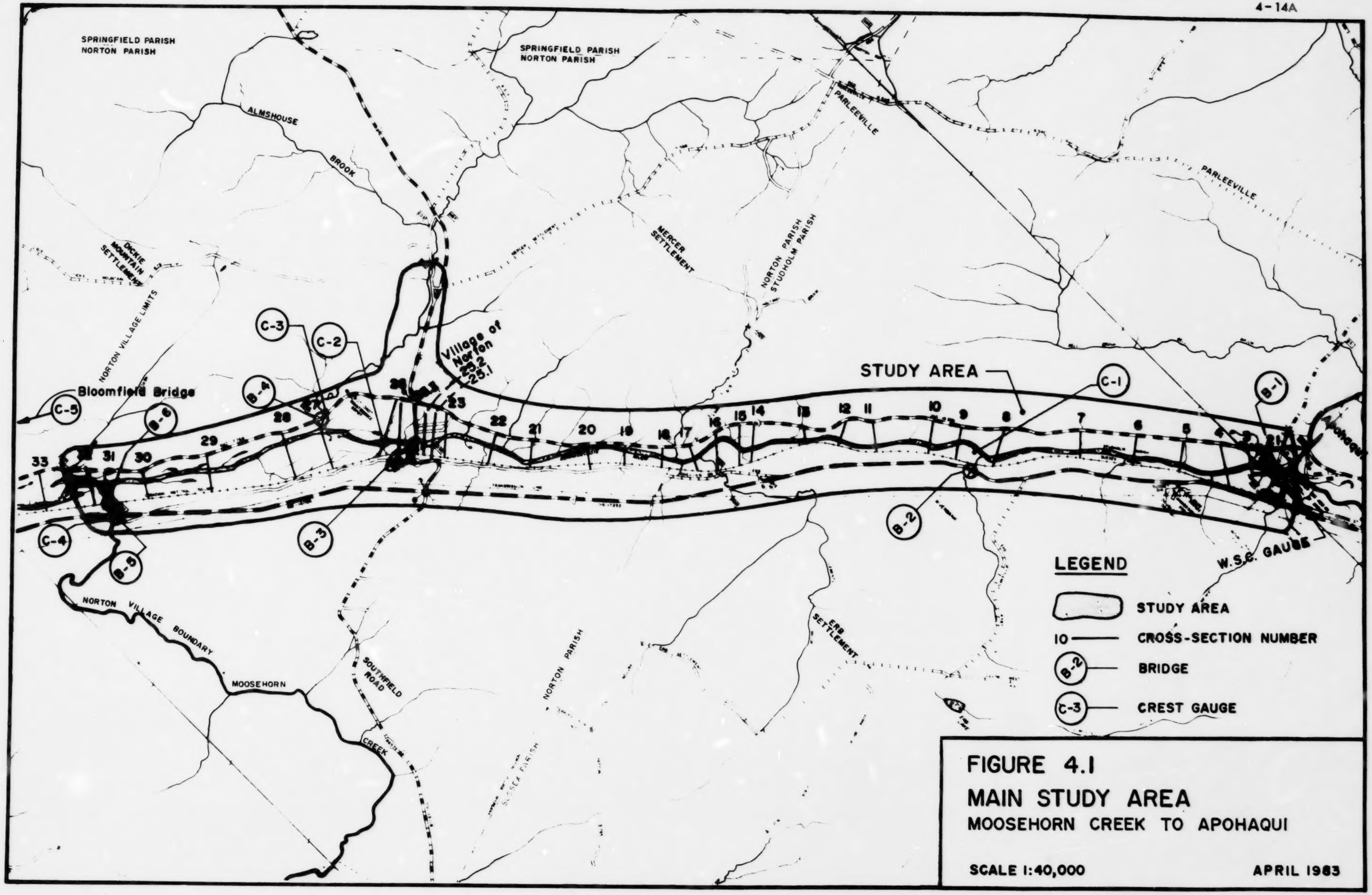
4.5 Summary of Hydraulic Analysis

The HEC-2 model was selected for use in determining the flood profile for the 1:20 and 1:100 year flows for the study area. Field surveys were carried out to obtain 34 floodplain and 11 river cross-sections within the study area. Two additional sections were obtained downstream of the main study reach. Details of all waterway crossings from Hampton to Apohaqui on the main stream and tributaries were obtained.

The field derived sections were combined with information from 1:10000 orthophoto mapping and bridge drawings provided by New Brunswick Department of Transportation to develop HEC-2 models of the study area. Three types of model were developed: a detailed model of the study reach from Moosehorn Creek to Apohaqui, a coarse model from Kennebecasis Bay to the downstream study limit and four tributary models of the first 0.5 kilometres of Moosehorn Creek, Almshouse Brook, Halfway Brook and Clements Brook.

Based upon the coarse model of the area downstream of the main study reach, a sensitivity analysis of water levels within the study reach to water levels on the Saint John River was carried out. For a wide range of levels on the Saint John River (i.e. from 1.0 m to 4.4 m GSC) the variation in water levels at Moosehorn Creek was only 0.04 m for both 1:20 and 1:100 year flows. Hence the choice of downstream level does not appear to be critical and the long term mean level for the Saint John River at Oak Point of 1.55 m GSC was selected.

Because of the empirical nature of the HEC-2 model it was appropriate to calibrate the developed model to the study reach. Two events, a bankfull flow and a 1:15 to 1:17 year flow, were used for this process. Manning's roughness coefficient for the channel and floodplain were adjusted to obtain satisfactory agreement between simulated and observed high water marks for these two events. Good agreement was achieved with an average absolute difference of 4 percent between observed and simulated water depths. Hence it was possible to utilize the model to simulate the 1:20 and 1:100 year flood profiles with considerable confidence.


SPRINGFIELD PARISH
NORTON PARISH
SPRINGFIELD PARISH
NORTON PARISH
ALMSHOUSE
BROOK
PARLEEVILLE
PARLEEVILLE
DICKIE MOUNTAIN SETTLEMENT
MERCER SETTLEMENT
NORTON PARISH
STUDHOLM PARISH
NORTON VILLAGE LIMITS
Village of Norton
23.2
23.1
STUDY AREA
Bloomfield Bridge
C-5
C-4
C-3
C-2
C-1
B-6
B-5
B-4
B-3
B-2
B-1
33
32
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11
10
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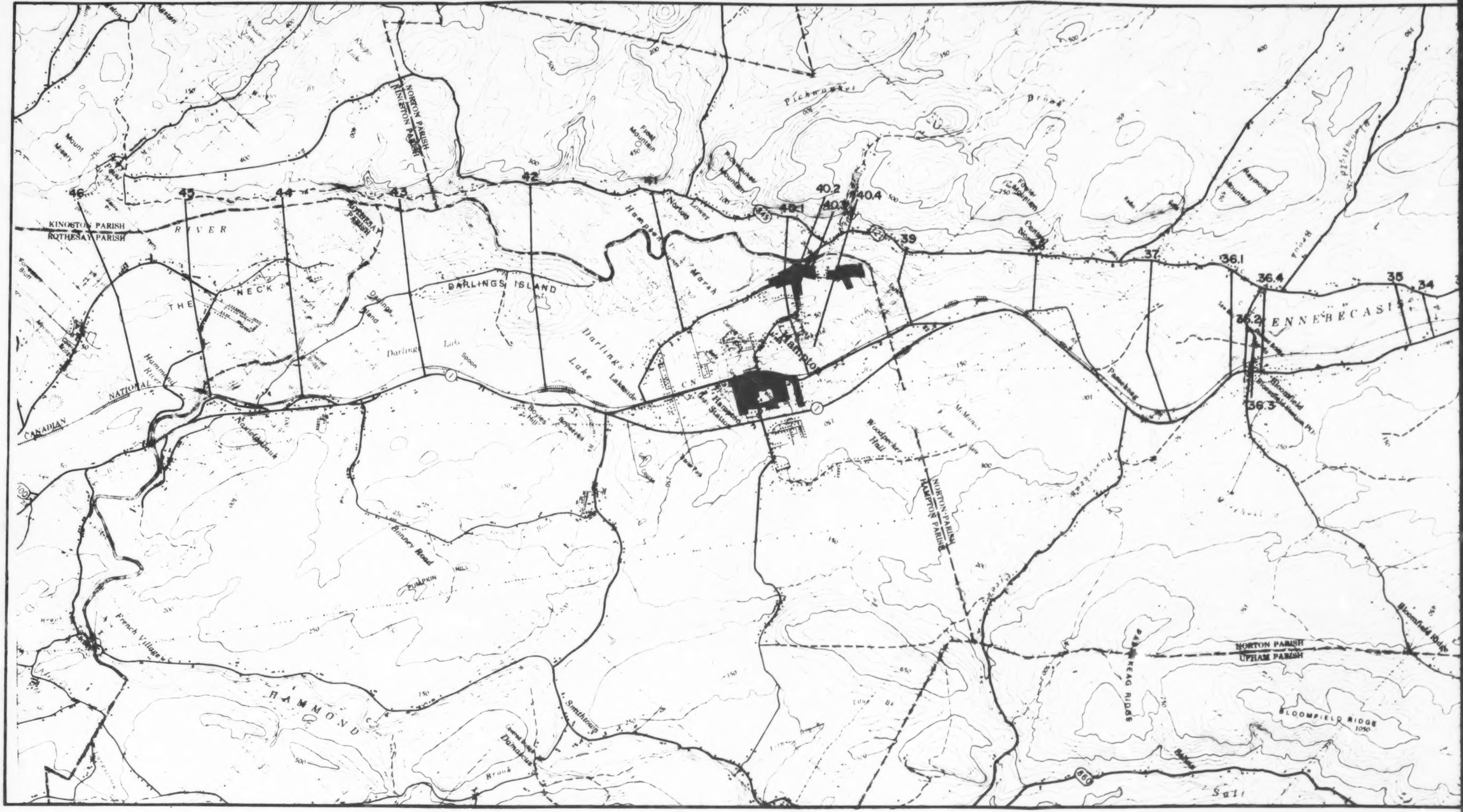
8
7
6
5
4
3
2
1
Apohaqui
W.S.C. GAUGE
NORTON VILLAGE BOUNDARY
MOOSEHORN
CREEK
SOUTHFIELD ROAD
NORTON PARISH
SUSSEX PARISH
ERB SETTLEMENT
LEGEND
STUDY AREA
10
CROSS-SECTION NUMBER
B-2
BRIDGE
C-3
CREST GAUGE

FIGURE 4.1
MAIN STUDY AREA
MOOSEHORN CREEK TO APOHAQUI

SCALE 1:40,000

APRIL 1983


46
45
44
43
42
41
40.2
40.1
40.3
40.4
39
38
37
36.1
36.4
36.2
36.3
35
34
KINGSTON PARISH
ROTHESAY PARISH
NORTON PARISH
KINGSTON PARISH
ROTHESAY PARISH
RIVER
THE NECK
DARLINGS ISLAND
Darlings Island
Darlings Lake
Lakeside
Hampton
Hampton Station
CN
KENNEBECASIS
CANADIAN NATIONAL
Hammond River
Mount Misery
Frost Mountain
Pickwauket Brook
Norton
Lower
Hampton Marsh
Woodpecker Hall
Bonney Road
PUMPKIN HILL
French Village
HAMMOND
Smithtown
Damascus
Brook
NORTON PARISH
HAMPTON PARISH
NORTON PARISH
UPHAM PARISH
PASSEKEAG RIDGE
BLOOMFIELD RIDGE
1050
Bloomfield Ridge
Bloomfield
Salt
Passekeag
1
100
860

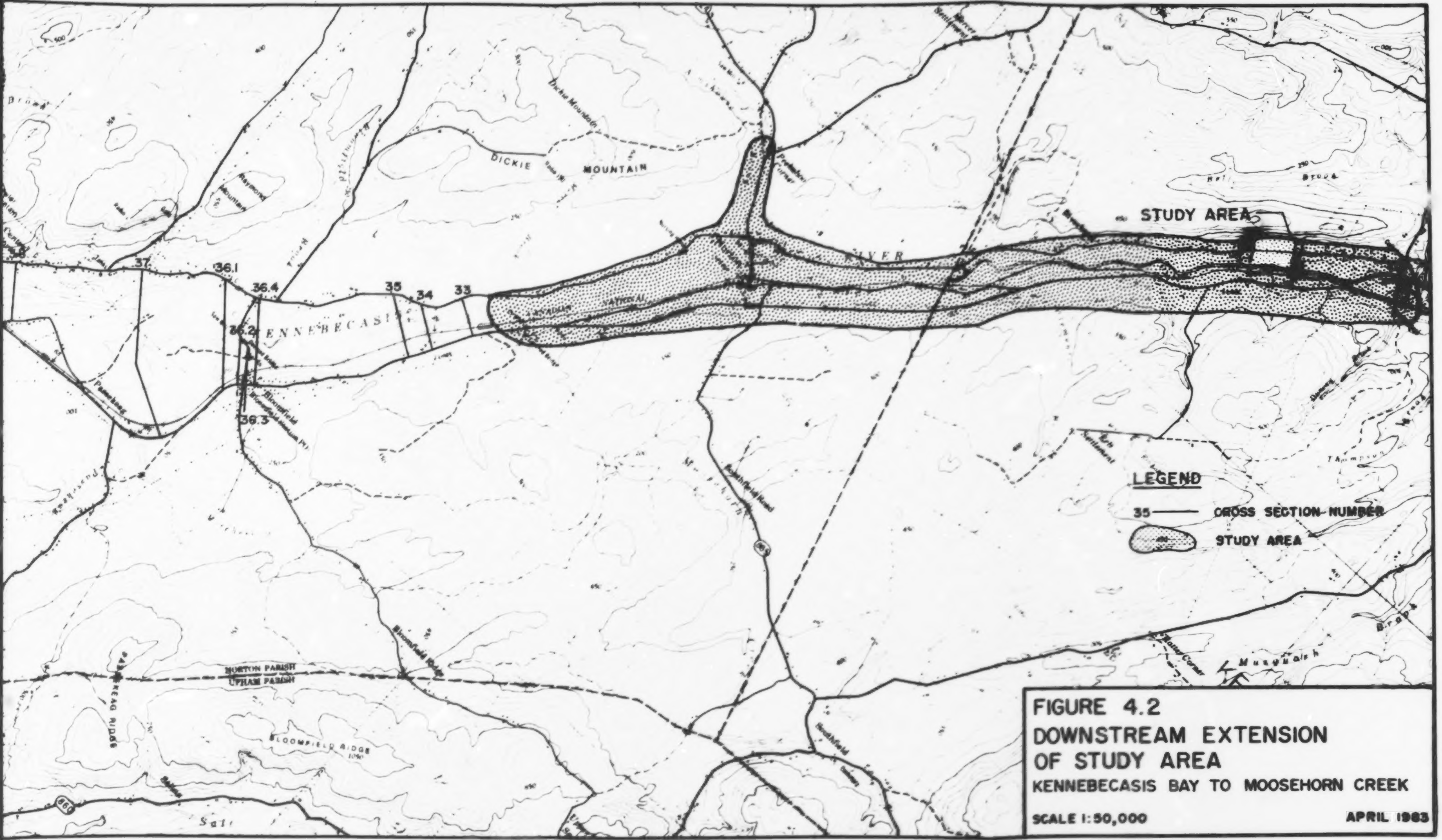

STUDY AREA
DICKIE MOUNTAIN
Dickie Mountain
KENNEBECASIS
RIVER
37
36.1
36.4
36.2
36.3
35
34
33
Bloomfield
NORTON PARISH
UPHAM PARISH
BLOOMFIELD RIDGE
1050
Thompson
Musquash
LEGEND
35 CROSS SECTION NUMBER
STUDY AREA

FIGURE 4.2
DOWNSTREAM EXTENSION
OF STUDY AREA
KENNEBECASIS BAY TO MOOSEHORN CREEK

SCALE 1:50,000 APRIL 1983

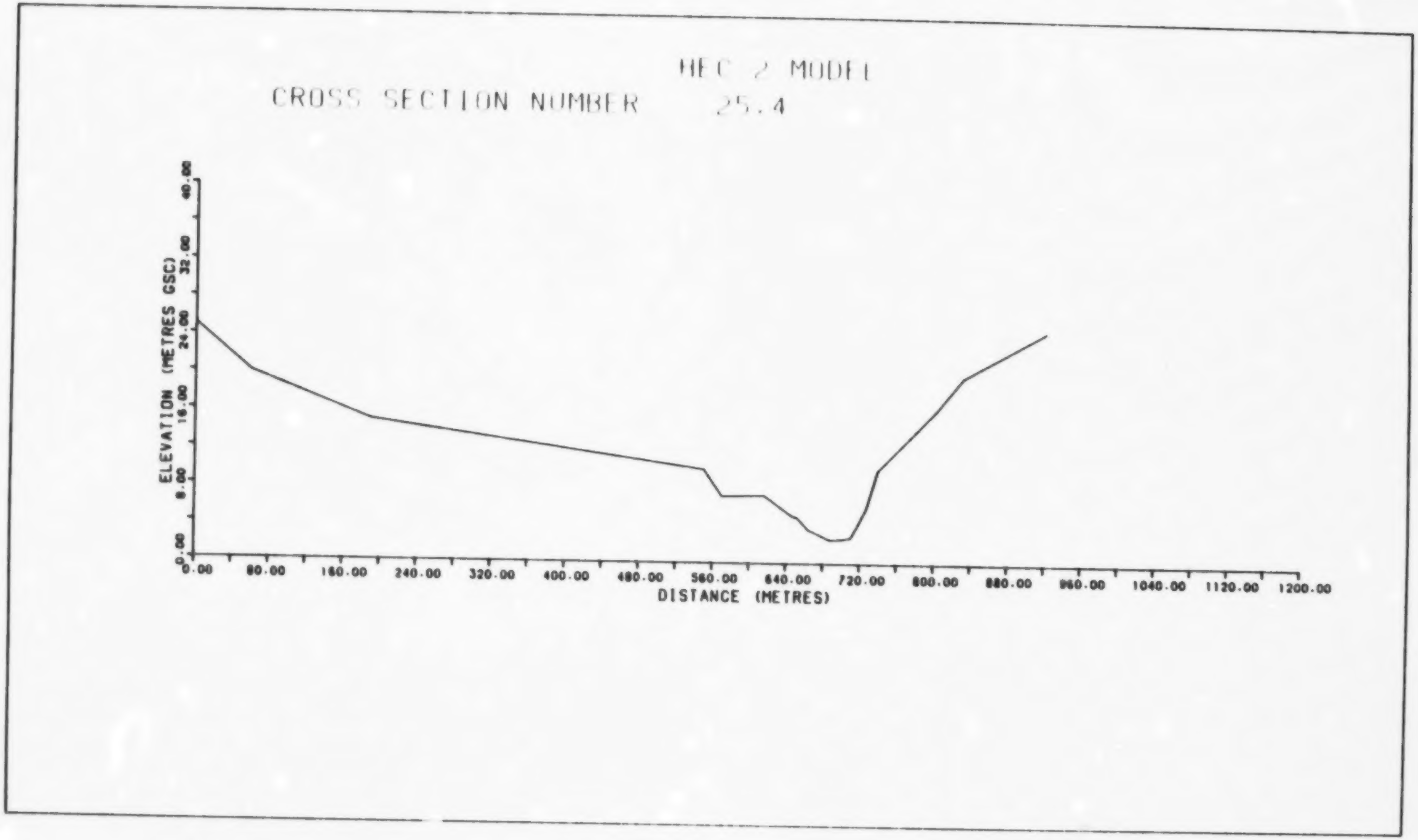


Figure 4.3: Typical HEC-2 Flood Plain and River Cross-Section (Norton Village)

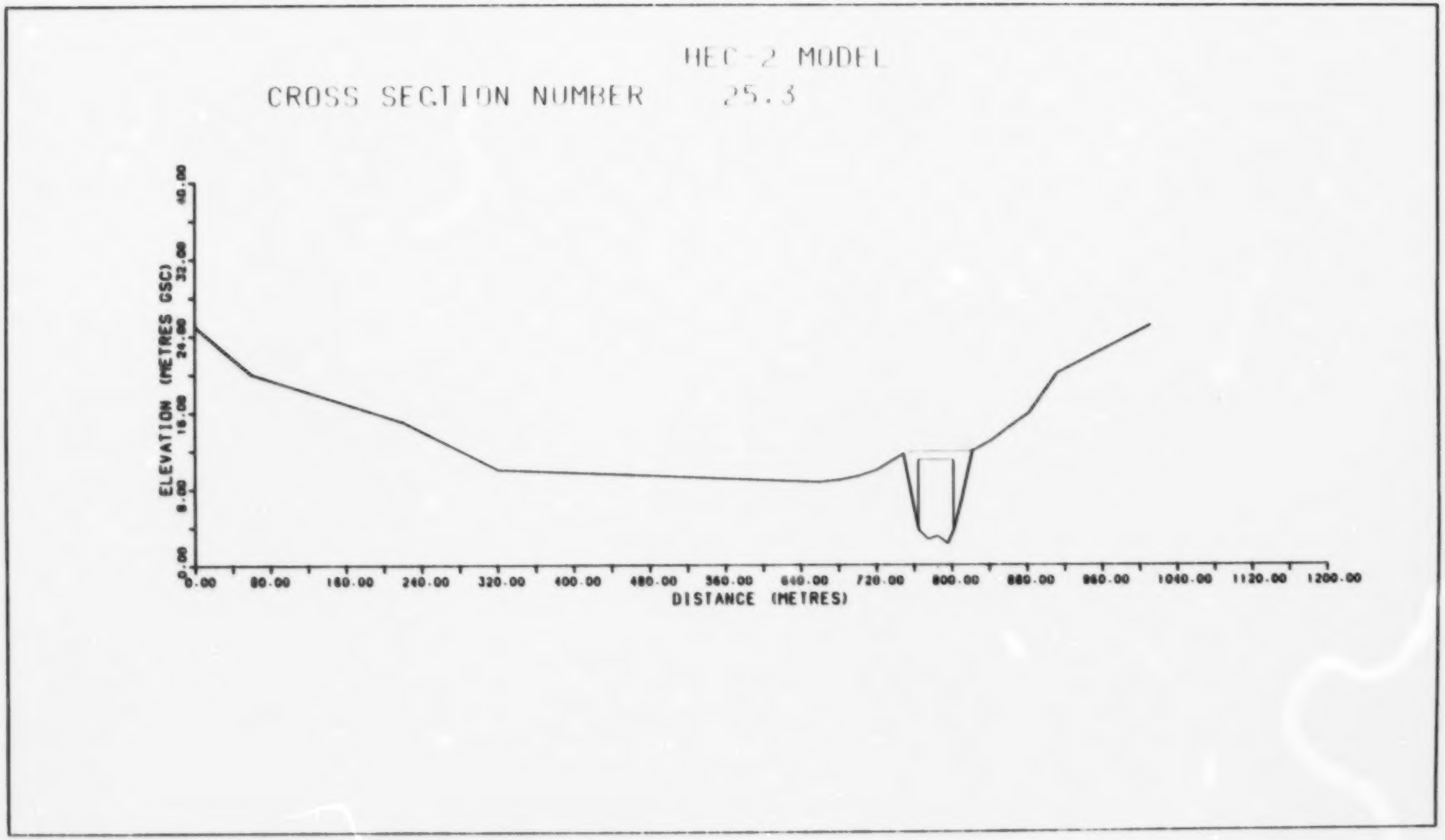


Figure 4.4: Typical HEC-2 Cross-Section at a Structure (Norton Bridge)

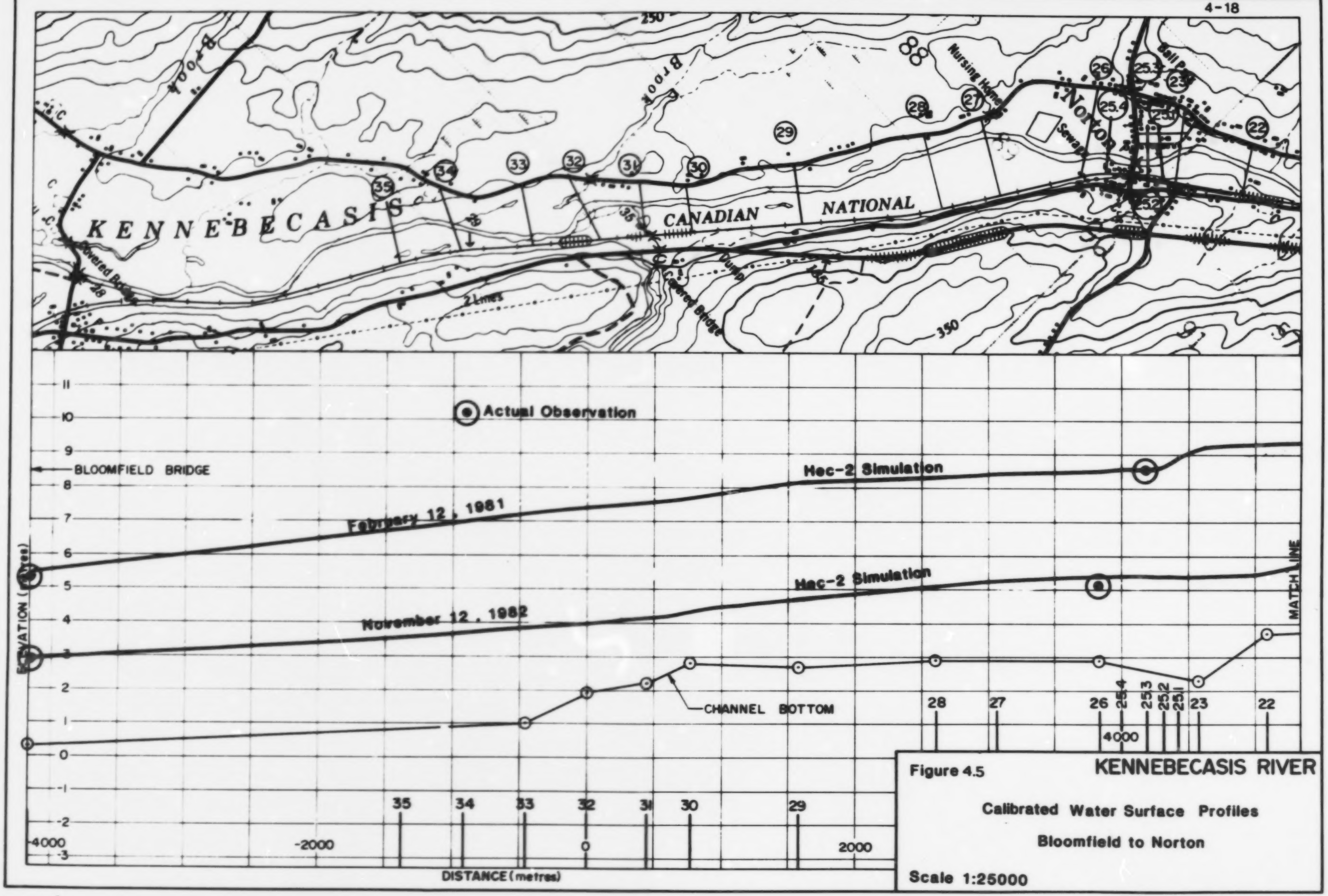


Figure 4.5

KENNEBECASIS RIVER

Calibrated Water Surface Profiles

Bloomfield to Norton

Scale 1:25000

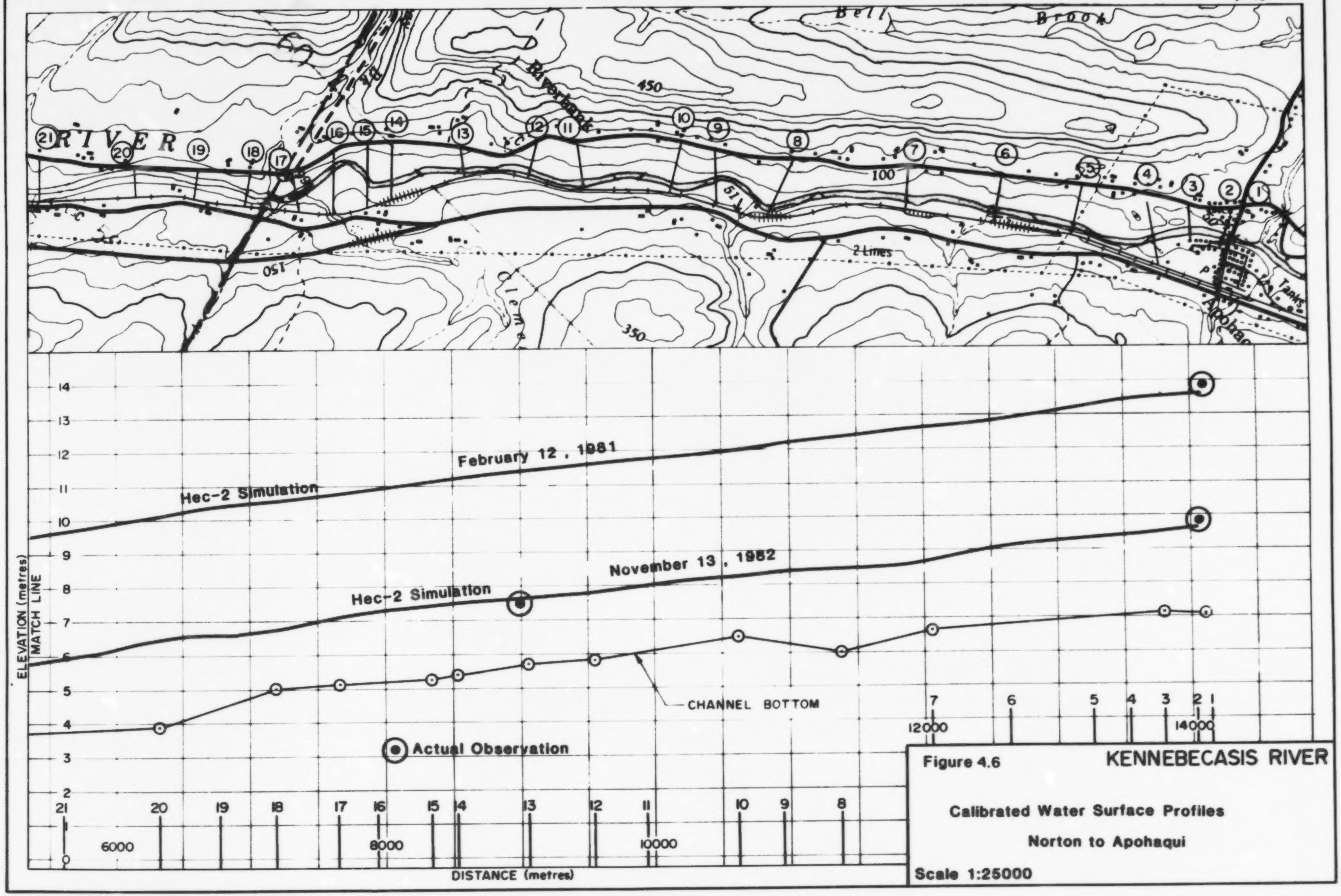


Figure 4.6

KENNEBECASIS RIVER

Calibrated Water Surface Profiles

Norton to Apohaqui

Scale 1:25000

5.0 DESIGN FLOOD WATER LEVELS

5.1 1:20 and 1:100 Year Water Levels

The flood level profiles for the 1:20 and 1:100 year design flows have been computed using the previously described calibrated HEC-2 models of the study area. Table 5.1 shows the resulting water levels. Figures 5.1 and 5.2 show the profiles for the main study reach.

The 1:100 year profile is generally above the historically well documented floods of 1962 and 1981. As the high water marks shown on Figures 5.1 and 5.2 indicate, the 1:100 year water level is between 0.6 m below and 0.4 m above the observed levels for the 1962 event. A 1:100 year flow would result in levels of about 1.0 m above those observed for the February 12, 1981 flood. The 1:20 year profile is very similar to that for the 1981 event. The observed high water marks are within 0.1 to 0.4 m and within 0.1 m of the 1:20 year profile at Norton and Apohaqui respectively.

A comparison of water levels on the four tributaries considered from the HEC-2 models of these reaches with the water level in the main river is presented in Table 5.2. For the first 0.5 km of each tributary the backwater from the main river is dominant for all cases for most of this length. In some cases, the tributary water level is slightly higher at the end of the 0.5 km reach than the Kennebecasis River level. Hence this elevation should be used to define the flood risk area at these points.

TABLE 5.1

FINAL WATER LEVELS FOR 1:20 AND 1:100 YEAR OPEN WATER CONDITIONS FOR THE KENNEBECASIS RIVER

	Design Water Level in Metres (GSC datum)		
Section No.	1:20 Year Return Period	1:100 Year Return Period	Location on Kennebecasis River
32.0	7.24	8.09	Moosehorn Creek
31.0	7.50	8.36	
30.0	7.69	8.54	
29.0	8.09	9.00	
28.0	8.47	9.41	
27.0	8.63	9.58	
26.0	8.75	9.70	Norton at
25.4	8.84	9.77	Railway Bridge
25.3	8.79	9.70	Norton Old
25.2	8.78	9.67	Highway Bridge
25.1	9.22	10.30	
23.0	9.45	10.59	
22.0	9.58	10.71	
21.0	9.89	11.00	
20.0	10.23	11.32	
19.0	10.52	11.61	
18.0	10.63	11.72	Halfway Brook
17.0	11.07	12.25	Clements Brook
16.0	11.18	12.38	
15.0	11.28	12.47	
14.0	11.36	12.53	
13.0	11.52	12.68	
12.0	11.81	12.96	
11.0	11.98	13.13	
10.0	12.21	13.37	
9.0	12.31	13.48	Secord Brook
8.0	12.66	13.84	
7.0	12.87	14.06	
6.0	13.19	14.39	
5.0	13.52	14.74	
4.0	13.72	14.94	
3.0	13.80	15.04	
2.0	13.89	15.11	Apohaqui
1.0	13.89	15.11	

TABLE 5.2

COMPARISON OF TRIBUTARY FLOOD LEVELS WITH MAIN STREAM BACKWATER LEVELS FOR 1:20 AND 1:100 YEAR DESIGN FLOWS

		1:20 Year Return Period		1:100 Year Return Period	
Tributary	Location	Water Level on Tributary	Kennebecasis Water Level	Water Level on Tributary	Kennebecasis Water Level
		(m)	(m)	(m)	(m)
Moosehorn Creek	Mouth	4.20	7.50	4.20	8.36
	CNR	5.81	7.50	6.02	8.36
	500 m limit	7.68	7.50	8.03	8.36
Almshouse Brook	Mouth	5.20	8.63	5.20	9.58
	Highway 21	7.17	8.63	7.74	9.58
	500 m limit	8.60	8.63	9.72	9.58
Halfway Brook	Mouth	7.80	11.07	7.80	12.25
	500 m limit	12.11	11.07	12.46	12.25
Clements Brook	Mouth	7.20	11.18	7.96	12.38
	500 m limit	10.24	11.18	10.29	12.38

5.2 Sensitivity Analysis of Design Levels

The computation of flood profiles is subject to various sources of uncertainty generally due to limited data. Hence, it is appropriate to examine the sensitivity of the final design profiles to variations in inputs and HEC-2 model parameters. The following analyses were carried out as part of this study:

- flows of 20 percent greater and less than the design flows for the 1:20 and 1:100 year conditions were simulated with the calibrated HEC-2 model of the study reach. This range was chosen since it was approximately equivalent to the difference between the various estimates of the design flows at Apohaqui shown in Table 3.1.

- using the final design flows, simulations were carried out with channel roughness of 20 percent greater and less than the calibrated values. This range was chosen since it covers a range of channel characteristics from a very smooth regular section with hard bed to a natural channel with irregular slopes and shape.

- using the final design flows, simulations were carried out with both channel and floodplain roughness increased and decreased by 20 percent of the calibrated values.

Table 5.3 shows the results of the sensitivity analysis with respect to the design flows at various points within the study reach. The average variation in levels is ±0.53 m and ±0.71 m in response to ±20 percent variations in the 1:20 and

1:100 year flows. This is equivalent to about a ±10% variation in flow depth. Variations in the positive and negative directions are approximately equivalent. The 1:100 year variations are larger than the 1:20 year since a 20 percent change in flow at this level produces a greater absolute change in flow.

Table 5.4 shows the results of the sensitivity analyses with respect to roughness of the channel and floodplain. Channel roughness was varied from a minimum of 0.024 to a maximum of 0.036 around a calibrated value of 0.030. For the floodplain within the study area the variation was from 0.14 to 0.21 compared to a calibrated value of 0.175. For an independent variation of the channel roughness the average change in levels is ±0.47 m and ±0.51 m for the 1:20 and 1:100 year flows respectively. The impact at both design levels is very similar and is equivalent to about a 7 to 8 percent variation in water depth. As Table 5.4 illustrates, variation of both the channel and floodplain roughness (both in the same direction) results in only a small change from the variation caused by varying the channel roughness alone. The average changes in response to a 20 percent increase or decrease in Manning's roughness coefficient are ±0.56 m and ±0.57 m for the 1:20 and 1:100 year cases. The design levels are therefore not very sensitive to quite wide variations in the floodplain roughness. This can be explained by the fact that on average only about 15 per cent of the total flow is carried by the floodplain. This results from the very low velocities (0.2 to 0.3 m/s) in this part of the cross-section because of the very large wetted perimeter.

TABLE 5.3

SENSITIVITY OF FLOOD PROFILES TO CHANGES IN FLOW

Location on Kennebecasis River	Section No.	Return Period	Design Flood Level (m)	Change in Level Due to Change in Flow (m) + 20% Flow	 - 20% Flow
Moosehorn Creek	32.0	20	7.24	+0.42	-0.21
		100	8.09	+0.60	-0.56
D/S Norton	26.0	20	8.75	+0.47	-0.44
		100	9.70	+0.59	-0.62
Norton	25.1	20	9.22	+0.55	-0.54
		100	10.30	+0.91	-0.69
Clements Brook	17.0	20	11.07	+0.59	-0.64
		100	12.25	+0.79	-0.76
Wilson Brook	9.0	20	12.31	+0.58	-0.62
		100	13.48	+0.72	-0.76
Apohaqui	2.0	20	13.89	+0.61	-0.67
		100	15.11	+0.72	-0.79

TABLE 5.4

SENSITIVITY OF FLOOD PROFILES TO CHANGES IN MANNING'S ROUGHNESS COEFFICIENT

Location on Kennebecasis River	Section No.	Return Period	Design Flood Level (m)	Change in Level Due to Change in Manning's n (m)			
				Channel Only		Channel and Overbank	
				N + 20%	N − 20%	N + 20%	N − 20%
Moosehorn Creek	32.0	20	7.24	+0.60	−0.40	+0.74	−0.47
		100	8.09	+0.75	−0.42	+0.55	−0.50
D/S Norton	26.0	20	8.75	+0.42	−0.40	+0.56	−0.49
		100	9.70	+0.50	−0.43	+0.54	−0.57
Norton	25.1	20	9.22	+0.42	−0.39	+0.49	−0.45
		100	10.30	+0.50	−0.35	+0.42	−0.44
Clements Brook	17.0	20	11.07	+0.39	−0.46	+0.51	−0.53
		100	12.25	+0.38	−0.47	+0.54	−0.58
Wilson Brook	9.0	20	12.31	+0.47	−0.54	+0.56	−0.61
		100	13.48	+0.51	−0.61	+0.63	−0.62
Apohaqui	2.0	20	13.89	+0.53	−0.62	+0.60	−0.66
		100	15.11	+0.59	−0.68	+0.70	−0.76

In summary it appears that water levels are sensitive in a non-linear manner to changes in both flow and Manning's roughness. A 20 percent variation in these factors produces about a 7 to 10 percent change in water depth. The absolute variations are of the order of 0.5 m which is equivalent to the contour interval of the mapping which will be used for delineating flood risk areas.

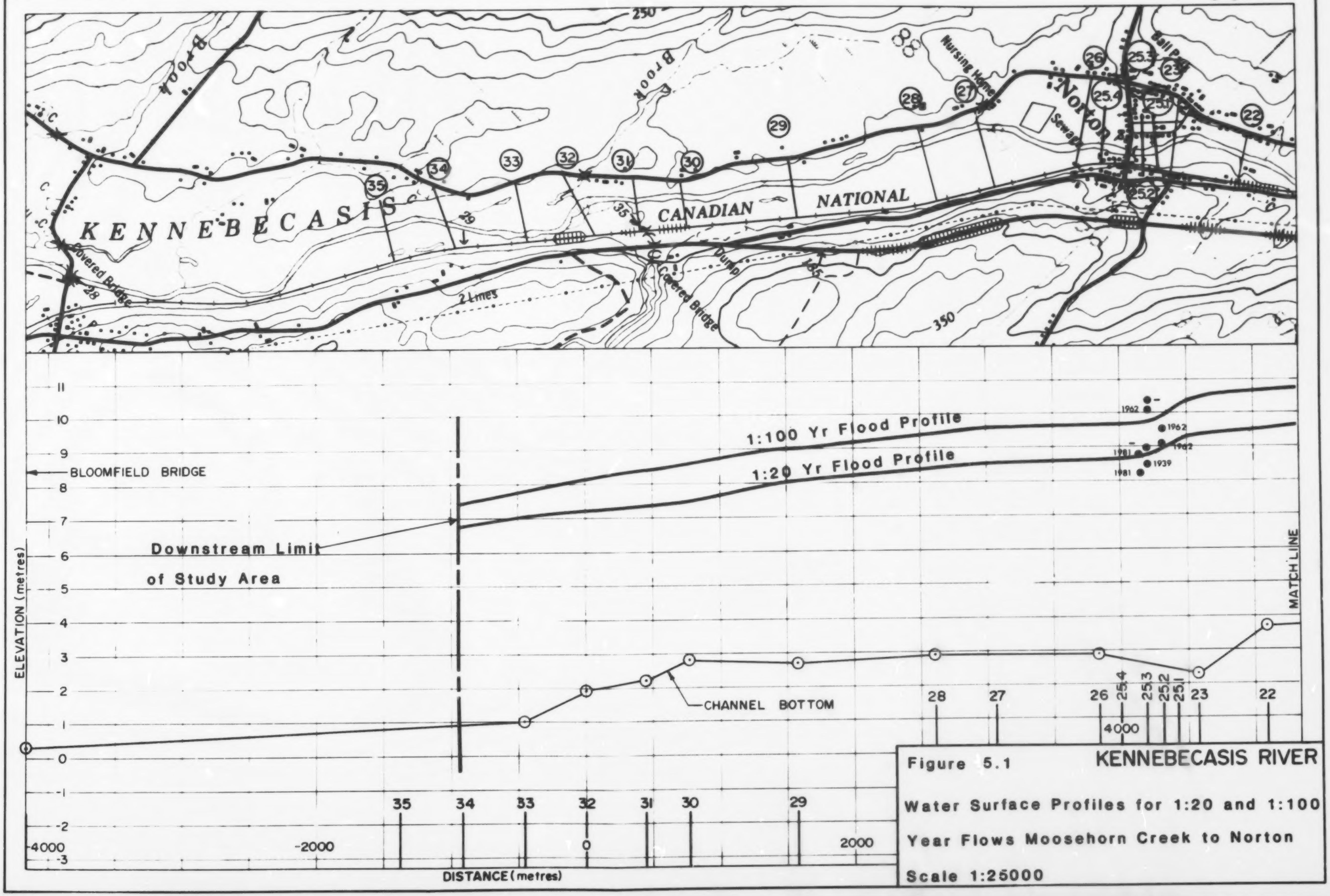

Brook
Brook
250
Nursing Home
Norton
Ball Park
Sewage
KENNEBECASIS
CANADIAN
NATIONAL
Covered Bridge
Covered Bridge
Dump
2 Lines
350
35
34
33
32
31
30
29
28
27
26
25.3
25.4
25.1
25.2
23
22
BLOOMFIELD BRIDGE
1:100 Yr Flood Profile
1:20 Yr Flood Profile
Downstream Limit of Study Area
CHANNEL BOTTOM
1962
1981
1939
ELEVATION (metres)
DISTANCE (metres)
-4000
-2000
0
2000
4000
MATCH LINE

Figure 5.1

KENNEBECASIS RIVER

Water Surface Profiles for 1:20 and 1:100 Year Flows Moosehorn Creek to Norton

Scale 1:25000

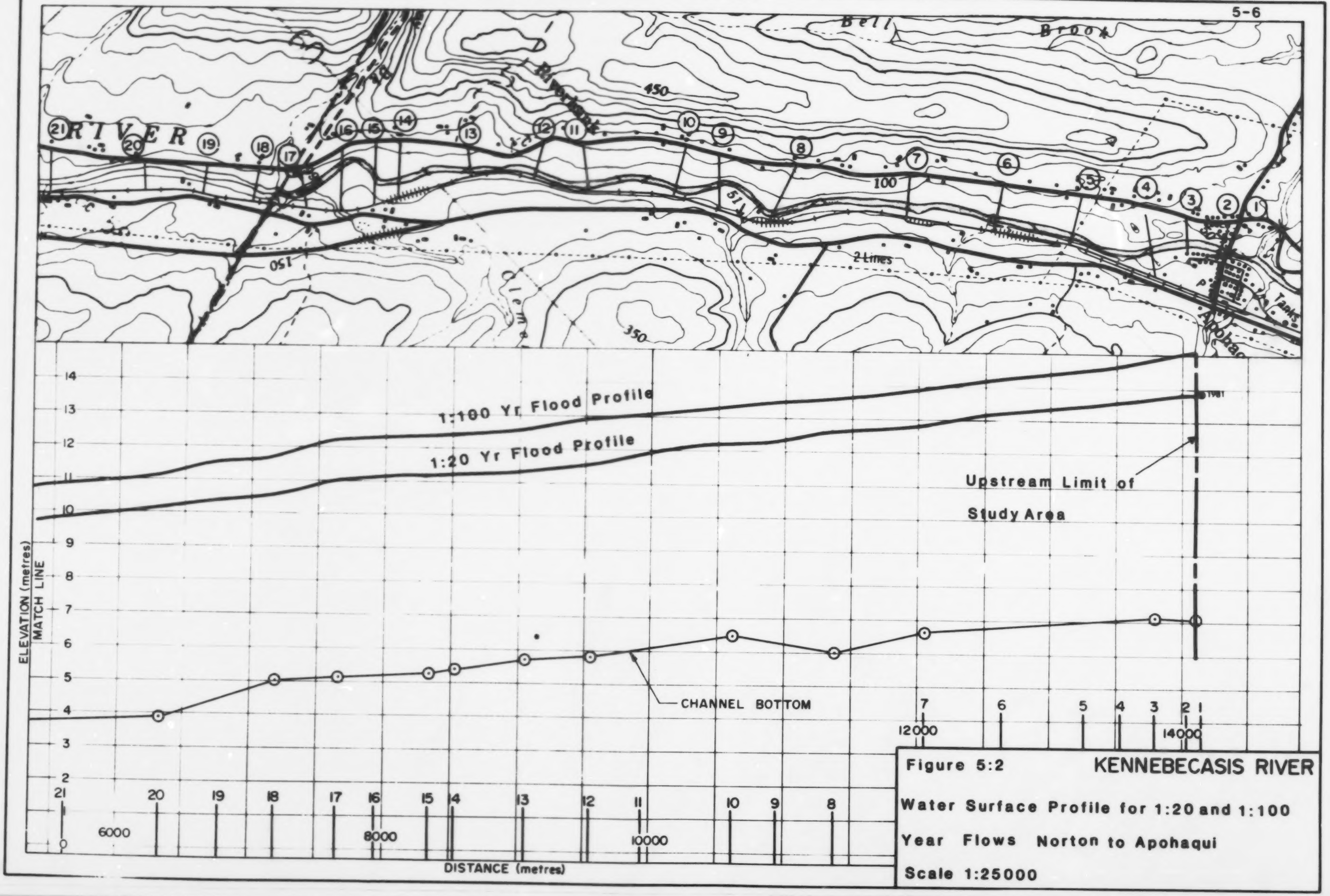


Figure 5:2 KENNEBECASIS RIVER

Water Surface Profile for 1:20 and 1:100 Year Flows Norton to Apohaqui

Scale 1:25000

6.0 ANALYSIS OF ICE CONDITIONS

6.1 Historical Observations of Ice Jams and Ice Characteristics

In order to carry out a complete analysis of ice jamming in the study area, there is a need for scientifically documented data on the characteristics of the river ice and of past ice jams. This should include:

- thickness and type of the ice
- size of ice flows
- location and type of ice jams
- length and thickness of ice jams
- measurements of ice jam roughness

In common with most similar situations, such data are not, for the most part, available for the study reach. Some information is available on the characteristics of the river ice from an observation program sponsored by the Canada - New Brunswick Flood Damage Reduction Program during the winters of 1980 to 1982. However, no similar measurements are available regarding specific ice jams. Hence, it was necessary to obtain as much information as possible through a review of historic flooding reports, interviews with local residents, a field reconnaissance and the measurement of observed ice scars.

The ice observation program sponsored by the Canada - New Brunswick Flood Damage Reduction Program consisted of regular monthly inspection trips by observers retained for the purpose. Four sites were visited on the Kennebecasis River in the study area. Additional inspections were also carried out during periods of particular interest by both the regular observers and by staff of Environment New Brunswick. Information collected included the extent of ice coverage, the type of ice, ice thickness and general conditions. Photographs of the ice cover were also obtained on some occasions. Ice thickness measurements taken mostly at Apohaqui varied from 0.2 to 0.6 m, averaging about 0.4 m.

A historic summary of flooding on the Kennebecasis River was presented in Table 2.3. This indicates that many of the documented events were related to the presence of ice jams. Jamming is mentioned as having caused flooding during the events of 1923, 1945, 1947, 1951, 1952 and 1953. Ice jams also occurred in connection with the flood events of 1902, 1909, 1936, 1970 and 1981 but in these cases the ice jam was not specifically mentioned as the primary cause of flooding. In particular, it appears that, for the 1981 event, ice jamming may have occurred downstream of Norton prior to the peak flow. However, by the time of peak flow, it had apparently cleared leaving peak water levels unaffected. Two sites are mentioned in these historic reports: at the confluence of the Kennebecasis River and Moosehorn Creek (i.e. at a point "two miles west of Norton") and at the old railway bridge in Norton.

Ice scars were observed throughout the main study area but were noticeably concentrated in the areas upstream of Moosehorn Creek and upstream of Norton. In general, these scars were at consistent elevations indicating depths of flow of about 4.5 to 5.5 m above their respective channel inverts. This is in the approximate region of a 1:5 to 1:20 year open water flood elevation. It is also relatively consistent with the estimated water level for the February 12, 1981 event. This was the last major flow to have occurred in the period prior to this study and would therefore be expected to be the source of the most recent scars.

6.2 Sites of Ice Jams

A local resident survey and field reconnaissance were conducted in March of 1983 to identify sites in the river that could initiate ice jams. Such sites could include sharp river bends, sudden contractions in river width, bridge abutments and piers, etc. Elevations of ice scars on structures and vegetation were also noted as an aid in the determination of the magnitude of an ice jam.

Four potential ice jam sites were identified within the study area as shown on Figure 6.1:

- at the confluence with Moosehorn Creek about 4 km downstream of Norton

- at the confluence of Almshouse Brook about 0.5 km downstream of Norton

- at the old railway bridge within the village of Norton
- at the confluence with Clements Brook about 2 km upstream of Norton

The first ice jam initiation site identified within the area is located at the bend in the Kennebecasis River at the confluences of the Kennebecasis River and Moosehorn Creek (see Figure 6.1). This reach presents a location where a consolidated ice cover would have sufficient strength to resist even high flows. An ice cover at this location would likely remain in place and contribute to initiating ice jams on a frequent basis, interrupted only by meteorological conditions that would cause it to weaken and decay prior to the spring freshette. Moreover, the bend and bars here would contribute to making this ice highly resistant to even a surge of ice and water released by the breakup of upstream jams. The potential for jams at this site may be further, although only slightly augmented by the influx of frazil or broken-up ice from Mercer Brook and Moosehorn Creek. However, for the most part, ice jams at this site are expected to be deep and initiated on the northwest bank of the bend.

Local residents mentioned that the ice on both Mercer Brook and Moosehorn Creek does in fact break-up before the ice in the main river. The ice flows out of the two tributaries and contributes to jamming at this site. It was said that flooding occurs as the result of an ice jam here about five out of every ten years. Historically, ice jams occurred here in 1945, 1947, 1951, 1952, and 1953 (see Table 2.3).

A shallow bar on the east bank of the Kennebecasis River just upstream of the confluence of Almshouse Brook is the second potential initiation site examined (see Figure 6.1). This site could initiate a jam and ice accumulation up through Norton. Local residents recall ice coming up onto the road on the north side of the river just below Almshouse Brook. It is this site that may have caused some flooding at Norton in 1970.

The third possible initiation site identified is the reach between the former railway bridge and the existing road bridge at Norton (see Figure 6.1). The river channel bends at a bottle neck below the road bridge and then carries through the remains of the railway bridge. Although the bridge deck and a portion of the central pier of the railway bridge were removed in 1975, the remaining abutments and central pier still form points on which an ice arch could develop. This could occur particularly if a downstream jam forms through this reach or a large flux of ice enters the area from the release of an upstream jam. Local residents mentioned that ice jams have always been a problem at the railway bridge. Historically documented ice jams occurred here in 1902, 1909, 1923, 1936, 1945, 1947, and 1981.

The river bank just upstream of the confluences of Halfway and Clements Brooks was identified as the fourth potential ice jam initiation site (see Figure 6.1). The relatively sharp bends would present an obstruction to the clear passage of ice and provide protection against breakup for the consolidated ice sheet on the western bend. Logs and other debris are lodged on the western bank, indicating that high flows would also force ice onto the bank.

Three of the four potential ice jam initiation sites within the study reach were considered significant in initiating ice jams capable of creating flood conditions on the Kennebecasis River. Of these three sites, site 1 at the confluence of Moosehorn Creek has perhaps the greatest potential for frequent and significant jamming. Site 3, at the old Norton railway bridge, was selected because the backwater effects of a jam here could be significant at Norton. Site 4, located at the confluences of Halfway and Clements Brooks, is an ideal site for frequent jamming and the receiving of ice released from ice jams upstream of Apohaqui.

Site 2, at the confluence with Almshouse Brook, was not selected as a significant initiation site due to the lack of ice jam evidence. Although ice has been reported on the road near the site it is more likely that this is the result of larger ice jams that would be created further downstream at site one.

Sites within the study area that may initiate ice jams are not the only important considerations. Ice jams may occur downstream of the study area and create flooding that backs up into it. The field reconnaissance therefore started by looking at the Kennebecasis River downstream of the study area before moving upstream through it. From the field reconnaissance it was apparent that the numerous meanders and the reverse bend below Bloomfield will obstruct the clearing of ice from the river and contribute to regular flooding below Bloomfield. However, the wide overbanks below Bloomfield and the conveyance in these overbanks should dampen the ice jam backwater effects upstream of Bloomfield. There appears to be some jamming potential at Bloomfield Bridge and at the river curves one to two kilometres upstream. The absence of physical evidence of frequent high water levels, however, suggests that the project study reach is virtually independent of ice effects in the area below it.

There are a number of bends, narrows, shallows and tributary confluences upstream of the study area. Their configurations suggest that frequent, although perhaps temporary ice jams occur here. The possibility of a surge of ice and flood wave from these sites moving downstream into the study reach is high. This may contribute to premature or dry ice jams forming within the study reach.

6.3 Analysis Using HEC-ICE Option of the HEC-2 Model

Upon selection of the initiation sites to be studied, their ice jam effects on the river water levels were studied using the ICE option of the HEC-2 backwater model[14]. This option allows the establishment of an ice cover on the river and computes the water levels that would result. It also allows for the simulation of an ice jam and computation of the resulting water levels.

The three selected jam sites were modelled individually because the occurrence of large simulataneous ice jams within the study area is unlikely. To determine the size of the ice jams to be modelled, the amount of ice that would collect at any one jam would not likely be greater than the amount of ice available for jamming within the river through the study reach. Therefore, the volume of ice that may form through the study reach during a normal flow period was used as the volume of ice that would create an ice jam. This volume of 360,000 m^3 was calculated by multiplying the study reach length of 16 km by an average ice thickness of 0.4 m1) and channel width of 60 m.

This volume of jam ice was distributed in the channel and overbank area of the initiation sites to simulate an ice jam. For river flows of bankfull, 1:20 and 1:100 year (see Tables 3.2 and 3.3 for the actual values), the ice jams were modified by adjusting jam thickness and length, while maintaining volume, until the ice jam became stable in the HEC-2 model. Reaching stability in HEC-2 indicates that conditions are such that an ice cover of the size modelled would be able to exist under those flow conditions.

This procedure was performed at all three sites for the three flows. Ice stability was reached and resulting water levels were accepted as being representative of expected ice jam conditions under these conditions. Table 6.1 presents the characteristics of the ice jams at each of the modelled locations for each flow considered.

---

1) This average thickness was derived from observations of ice thickness obtained by staff of Environment New Brunswick during the winters of 1980 to 1982.

TABLE 6.1

CHARACTERISTICS OF MODELLED ICE JAMS REQUIRED FOR STABILITY FOR VARIOUS FLOWS

	Site 1			Site 3			Site 4		
Characteristics	Bankfull Flow	1:20 Yr Flow	1:100 Yr Flow	Bankfull Flow	1:20 Yr Flow	1:100 Yr Flow	Bankfull Flow	1:20 Yr Flow	1:100 Yr Flow
Thickness (m) in Channel	1.0-2.5	2.0	2.0-2.2	1.0-1.5	3.0	3.0	1.0	2.0	2.0-2.1
Length of Jam (m) in Channel	2000	3100	3100	4300	2900	2900	3100	2700	2700
Thickness (m) in Overbank	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Length of Jam (m) in Overbank	2000	500	500	4300	300	300	3100	1200	1200
Roughness of Jam 1)	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05	0.05

1) This value was obtained during simulations carried out to achieve a stable jam and represents a composite value for channel and overbank ice. A range of values from 0.03 to 0.08 was examined.

6.4 Impact of Ice Jams on Flood Water Levels

Upon completion of the modelling of the three potential ice jams and their resulting water levels, backwater levels for ice jams and open water were compared. Table 6.2 is a brief summary of the maximum increase of the open water level due to the modelled ice jams. Also shown is the distance upstream from the jam site that experiences an increase in the water level due to an ice jam. These values are given for the three jam sites and the three river flows used. The increase in water levels ranged from a minimum of 1.33 m for the 1:100 year at site 3 to a maximum of 2.03 m for the 1:20 year at site 4. The affected reaches were quite substantial with none being less than six kilometres and some extending beyond the study area.

Two locations where the effect on water levels by ice jams is important are the communities of Norton and Apohaqui. An increase in open water levels due to ice jams at these locations will increase the amount of flood damage expected. Table 6.3 shows increases in open water levels at Apohaqui and two locations in Norton, upstream and downstream of the road bridge, due to ice jams. Considering all ice jam site and flow combinations, the maximum increase in open water levels at both Norton and Apohaqui due to ice jams is in the order of 1.5 m.

Figures 6.2 to 6.6 show comparisons of the open water and ice jam backwater levels for both the bankfull and 1:20 year flows. The 1:100 year levels were not plotted on these figures because the effects of ice jams on the 1:100 year open water levels is very similar to that for the 1:20 year

TABLE 6.2

EFFECT OF ICE JAMS ON WATER LEVELS

	Length of Reach Affected by Ice Jam Backwater (m)			Maximum Ice Jam Water Level Increase Over Open Water (m)		
Jam Initiation Site	Bankfull	1:20 Yr	1:100 Yr	Bankfull	1:20 Yr	1:100 Yr
1	9 500	11 500	>14 000 (upstream of Apohaqui)	1.72	1.84	1.90
3	10 000	>10 000 (upstream of Apohaqui)	>10 000 (upstream of Apohaqui)	1.82	1.55	1.33
4	>6 000 (upstream of Apohaqui)	>6 000 (upstream of Apohaqui)	>6 000 (upstream of Apohaqui)	1.75	2.03	1.93

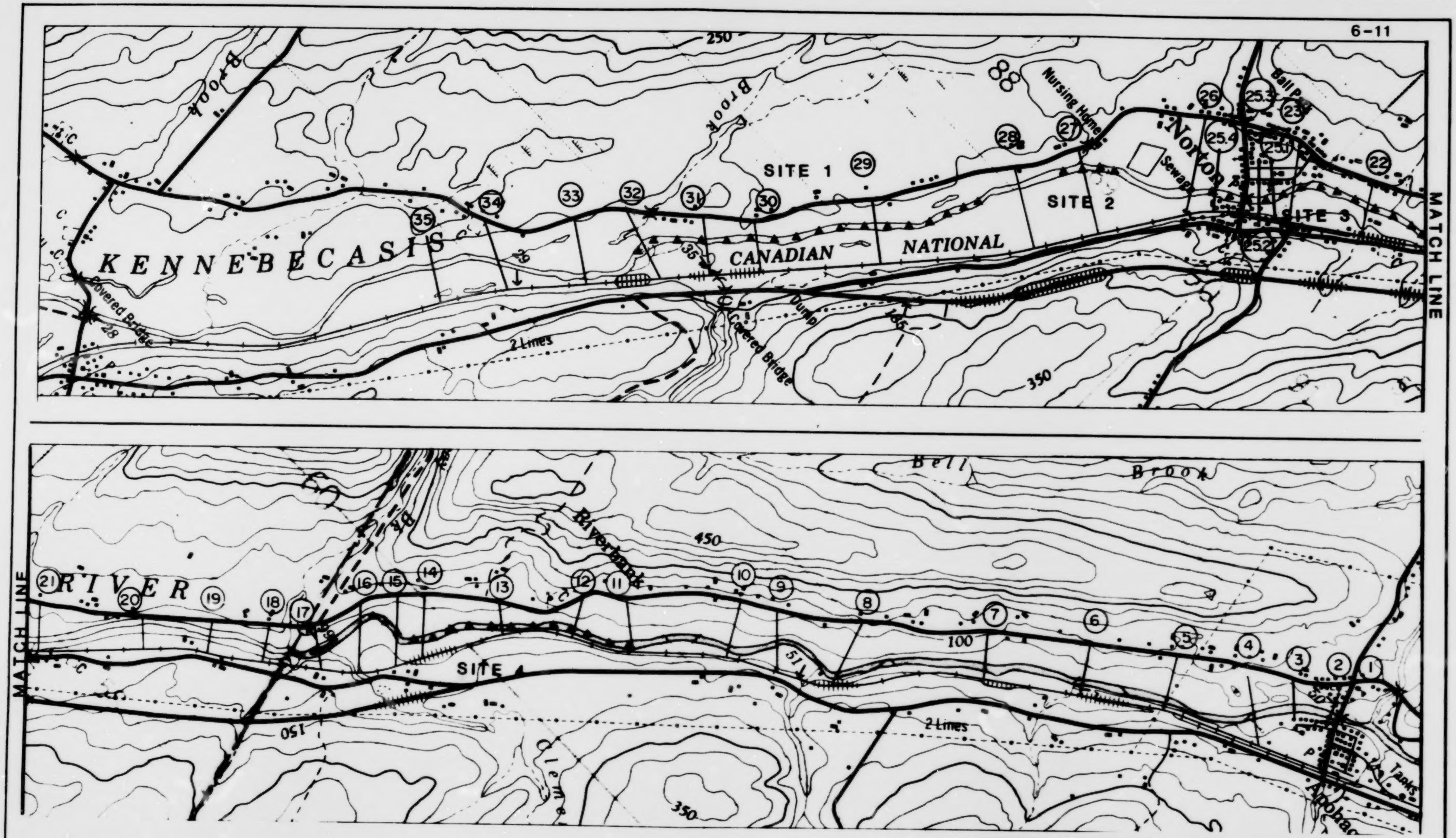
TABLE 6.3

INCREASE ABOVE OPEN WATER LEVELS AT NORTON AND APOHAQUI DUE TO ICE JAMS

	Bankfull Flow			1:20 Year Flow			1:100 Year Flow		
Jam Initiation Site	Norton d/s of Road Bridge	Norton u/s of Road Bridge	Apohaqui	Norton d/s of Road Bridge	Norton u/s of Road Bridge	Apohaqui	Norton d/s of Road Bridge	Norton u/s of Road Bridge	Apohaqui
1	0.97	0.87	0.03	1.41	1.11	0.09	1.50	0.99	0.10
3	1.31	1.44	0.10	0.92	1.31	0.19	0.80	1.08	0.19
4	-	-	0.66	-	-	1.03	-	-	1.01

condition. The ice jam lengths are very similar and the increase in open water levels due to the ice jams are also about the same for both the 1:20 and 1:100 year flows.

Figures 6.2 to 6.6 also indicate the elevations of observed ice scars within the study reach. In general these lie between the water level profiles for the bankfull flow and the 1:20 year flow with ice jams superimposed. With one exception none of the observed ice scars are above the 1:20 year ice jam profiles. As Table 2.3 indicates, there are several historical references to flooding at about 0.0 to 0.3 m above the main street in Norton in connection with ice jams. This gives a water level of about 9.0 m elevation which corresponds closely to the ice scars noted in that area. This would therefore appear to indicate that ice jams have tended to occur at relatively lower flows, i.e. between bankfull and the 1:20 year flow. For higher flows it appears that although theoretically the ice jams could be sustained (as indicated by the HEC-2 simulations) that in practice conditions have not been conducive to formation of a stable jam. This may be due to more rapid clearing of the ice under high flows reducing the available volume of ice for jamming. Hence it appears that although the HEC-2 simulations indicate maximum water levels in the study area would result from ice jams in combination with a 1:100 year design flow that in fact the 1:100 year open water profile is more likely the true 1:100 year flood profile. Ice jam water levels appear to be limited to elevations similar to the 1:20 year open water profile. This is borne out by the historical information in that maximum observed levels occurred in 1962 and 1981 without ice jam effect.


KENNEBECASIS
RIVER
CANADIAN
NATIONAL
SITE 1
SITE 2
SITE 3
SITE 4
MATCH LINE
MATCH LINE
Norton
Nursing Home
Sewage
Ball Park
Covered Bridge
Covered Bridge
Dump
2 Lines
2 Lines
Brook
Brook
Bell Brook
Riverbank
Apohaq
Tanks
250
350
350
450
150
100
35
36
28
165
115
1
2
3
4
5
6
7
8
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25.1
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32
33
34
35

LEGEND

▲ Ice Jam

Figure 6.1 KENNEBECASIS RIVER

Historical and Potential Ice Jam Sites
Moosehorn Creek to Apohaqui

Scale 1:25000

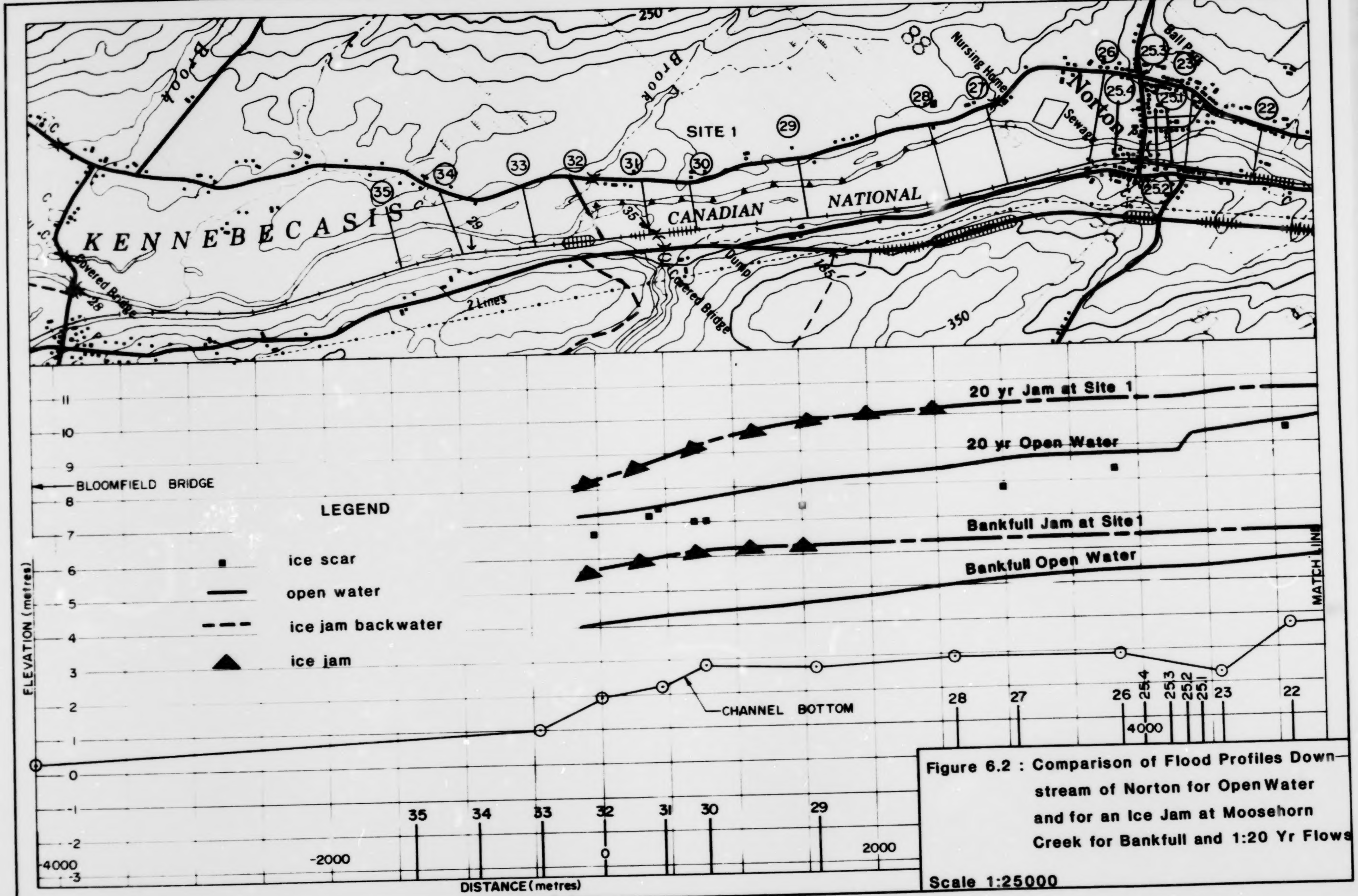
Brook
Brook
250
SITE 1
Nursing Home
Ball Park
Norton
Sewage
KENNEBECASIS
CANADIAN
NATIONAL
Covered Bridge
Covered Bridge
Dump
2 Lines
350
35
34
33
32
31
30
29
28
27
26
25.3
23
25.4
25.1
22
25.2
20 yr Jam at Site 1
20 yr Open Water
Bankfull Jam at Site 1
Bankfull Open Water
BLOOMFIELD BRIDGE
LEGEND
ice scar
open water
ice jam backwater
ice jam
CHANNEL BOTTOM
MATCH LINE
FLEVATION (metres)
DISTANCE (metres)
11
10
9
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4000
-2000
0
2000
4000

Figure 6.2 : Comparison of Flood Profiles Downstream of Norton for Open Water and for an Ice Jam at Moosehorn Creek for Bankfull and 1:20 Yr Flows

Scale 1:25000

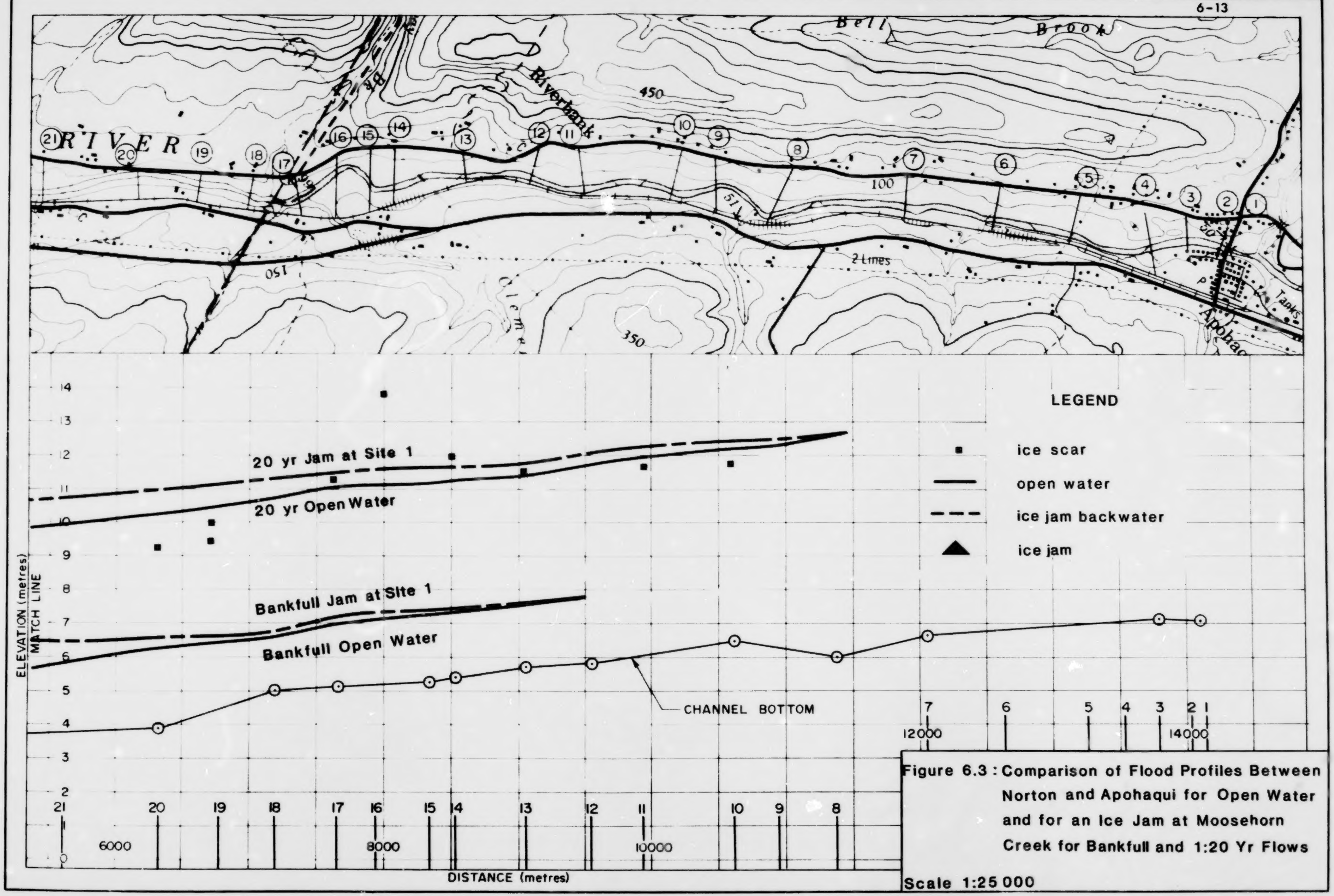

Figure 6.3 : Comparison of Flood Profiles Between Norton and Apohaqui for Open Water and for an Ice Jam at Moosehorn Creek for Bankfull and 1:20 Yr Flows

Scale 1:25 000

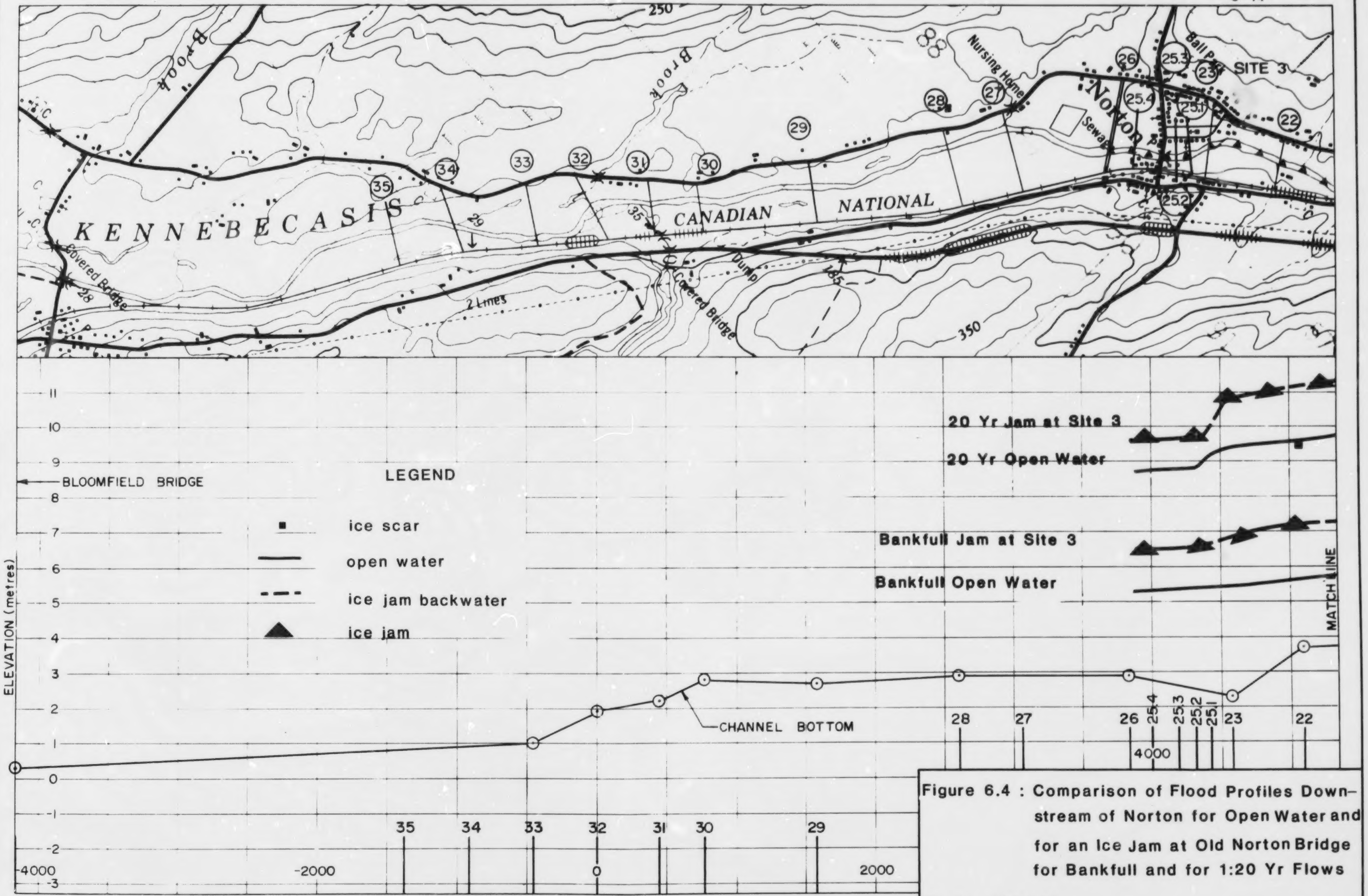

Figure 6.4 : Comparison of Flood Profiles Downstream of Norton for Open Water and for an Ice Jam at Old Norton Bridge for Bankfull and for 1:20 Yr Flows

Scale 1:25 000

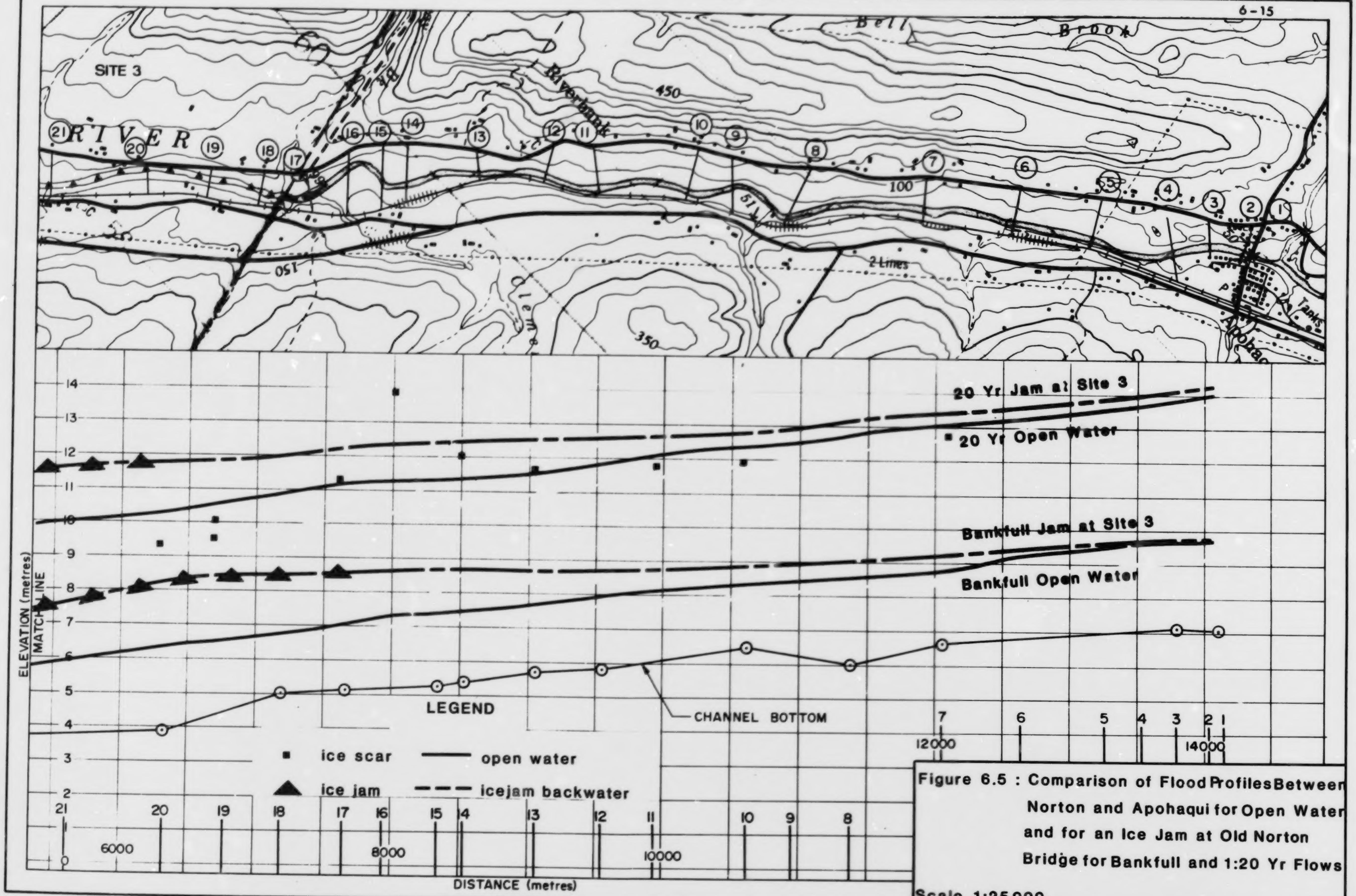

Figure 6.5 : Comparison of Flood Profiles Between Norton and Apohaqui for Open Water and for an Ice Jam at Old Norton Bridge for Bankfull and 1:20 Yr Flows

Scale 1:25000

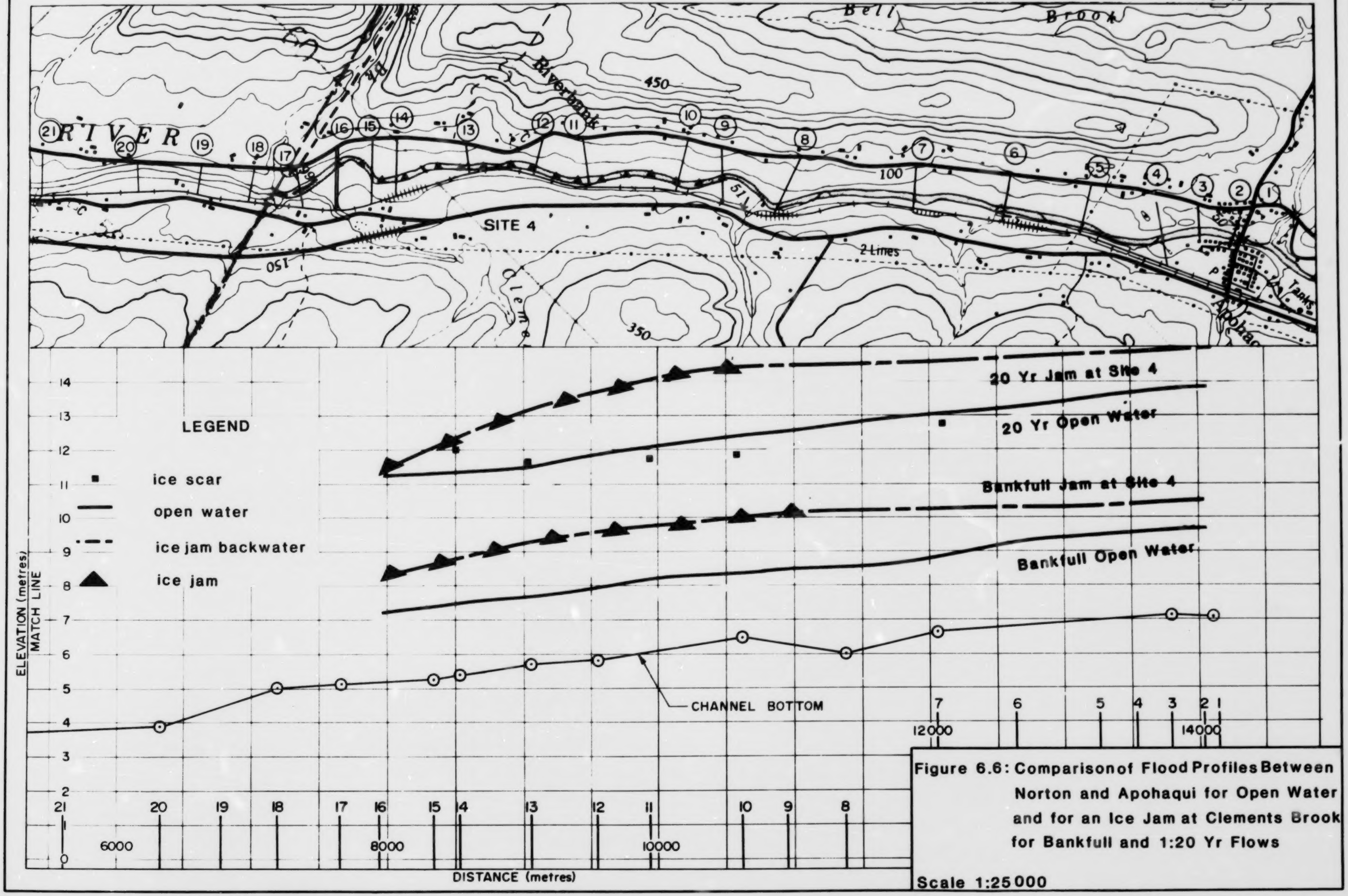

Figure 6.6: Comparison of Flood Profiles Between Norton and Apohaqui for Open Water and for an Ice Jam at Clements Brook for Bankfull and 1:20 Yr Flows

Scale 1:25000

7.0 CONCLUSIONS AND RECOMMENDATIONS

Historical evidence has indicated that the Kennebecasis River in the Norton area is prone to flooding on a relatively frequent basis. Hence a study of this area was initiated under the auspices of the Canada-New Brunswick Flood Damage Reduction Program in November 1982 to determine the 1:20 and 1:100 year flood profiles.

Water level profiles have been developed for the Kennebecasis River from Moosehorn Creek to Apohaqui for 1:20 and 1:100 year design flows and for the first 0.5 kilometres of four tributaries in this reach. The impact of ice jams at various locations has also been examined. The following sections provide the conclusions and recommendations from these studies.

7.1 Conclusions

i) It was concluded from an analysis of various previous studies[3,9] that design flows for the study area could be derived from the subregional frequency analysis performed during the Sussex area hydrotechnical study[3]. However, an updated frequency analysis of flow records for the Kennebecasis River at Apohaqui indicated a need to adjust the design flows from the subregional analysis to ensure compatibility with the 1:20 and 1:100 year flows at Apohaqui.

ii) It was concluded that flows for calibration purposes could be derived from the observed flow at Apohaqui adjusting for drainage area by means of the areal exponent of the Sussex subregional frequency relationship.

iii) From a sensitivity analysis of water levels in the study area to downstream levels on the Saint John River, it was concluded that over a wide range of downstream levels (1.0 to 4.4 m GSC) the effect at Moosehorn Creek was limited to a variation of 0.04 m. Hence an average water level of 1.55 m GSC was selected as the downstream condition for the design flow simulations.

iv) The HEC-2 model was calibrated using data from two well-documented high flow events: a bankfull flow on November 13, 1982 and the flow of record at the Apohaqui gauge of February 12, 1981. It was concluded that the modelled water levels agreed closely with the observed levels for both events. This was particularly true for the main study reach in which deviations between observed and simulated levels were between 0.01 and 0.17 m. For both events, the average absolute error was 4 percent of the observed depth.

v) The calibrated model was used to derive the 1:20 and 1:100 year flood profiles for the study area. The 1:100 year profile was approximately 1.0 m higher than that observed for the February 12, 1981 event. The 1:20 year profile is almost coincident with the latter profile.

vi) The effect of variations in the design flows and in Manning's roughness coefficients of the channel and floodplain on design flood profiles was examined. It was concluded that effects were non-linear with 20 percent variations leading to 7 to 10 percent changes in water depths. Absolute variations were of the order of 0.5 m which is equivalent to the contour interval of the mapping upon which the flood risk areas will be delineated.

vii) From historical evidence and field reconnaissance, it was concluded that four sites of potential or historical ice jamming exist within the main study reach. Of these, it was determined that one was not significant and hence the effect of ice jams at three sites, Moosehorn Creek, the old Norton railway bridge and Clements Brook, was examined. Simulations were carried out using the HEC-ICE option of the HEC-2 model superimposing jams upon a bank-full flow, a 1:20 year and a 1:100 year flow. It was concluded that water levels would be raised a maximum of 1.5 to 2.0 m above the open water levels at the jam site. This would result in maximum increases of about 0.8 to 1.5 m at Norton. From historical reports and observed ice scars, it was concluded that in practice jamming may not occur at the higher flows. It appears that flows of less than a 1:20 year flow have been associated with past jams. Hence a 1:100 year flood profile would correspond to the open water situation previously described.

7.2 Recommendations

i) On the basis of the analyses carried out in deriving the 1:20 and 1:100 year open water flood profile, it is recommended that the water levels presented in Tables 5.1 and 5.2 be used to delineate the 1:20 and 1:100 year flood risk areas when suitable topographic mapping becomes available.

ii) It appears that flooding of low-lying areas within the floodplain of the Kennebecasis River in the Norton area is almost an annual occurrence. For less frequent flows the area inundated also includes parts of the village of Norton. Since the total area of flooding is so extensive it is unlikely that remedial measures could be taken in a cost-effective manner to prevent all flooding. Some type of remedial works such as a ring-dyke might be used to protect Norton itself but the benefit-cost position requires further study beyond the scope of this investigation. Floodproofing or non-structural methods such as zoning, acquisition or flood-preparedness probably provide the only effective means of controlling flood damages.

iii) In addition to flooding from open water, flooding has been experienced due to ice jams at the confluence of Moosehorn Creek and at the old Norton railway bridge. The former location consists of an S-bend with several islands in the stream. It would appear that straightening and clearing this area might provide a reduction in the potential for ice jam initiation. The latter site is at a constriction in the river formed by the abandoned abutments of the old railway bridge. Removal of this constriction would probably reduce the chances of ice jam initation. However, in both cases the cost-effectiveness of these measures must be further examined which was beyond the scope of this study.

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Number/
Numéro

2

Norton Area Hydrotechnical Study
Kennebecasis River

Supplementary Report on Field Investigations

The Canada – New Brunswick Flood Damage Reduction Program

FENCO in association with
MACLAREN PLANSEARCH
and GEOCON

Lavalin

March 1984

The cover photograph shows the community of Norton inundated by floodwaters from the Kennebecasis River during the event of February 12, 1981. The photograph was taken by the staff of Environment New Brunswick.

NORTON AREA HYDROTECHNICAL STUDY
SUPPLEMENTARY REPORT ON FIELD INVESTIGATION

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1.0 INTRODUCTION

On October 27, 1982 Fenco Consultants Ltd., in association with MacLaren Plansearch and Geocon Inc., were retained by Environment New Brunswick to investigate the hydraulic characteristics of the Kennebecasis River in the Norton area with a view to delineating the limits of the flood hazard in this area. The study area consists of the flood plain and channel of:

a) The Kennebecasis River from the bridge at Apohaqui to a point 0.5 km downstream of the confluence of Moosehorn Creek with the Kennebecasis River.

b) All tributaries to the above to either the limit of the backwater effect from the 1:100 year flood or a distance of 0.5 km upstream from their confluence. The two major tributaries of the Kennebecasis River in this area are Almshouse Brook and Moosehorn Creek.

The limits of the study area are shown in Figure 1-1.

To accomplish this, the HEC II computer model was employed to delineate the open water surface profiles for the 1:20 and 1:100 year recurrence interval floods. The effects of flooding

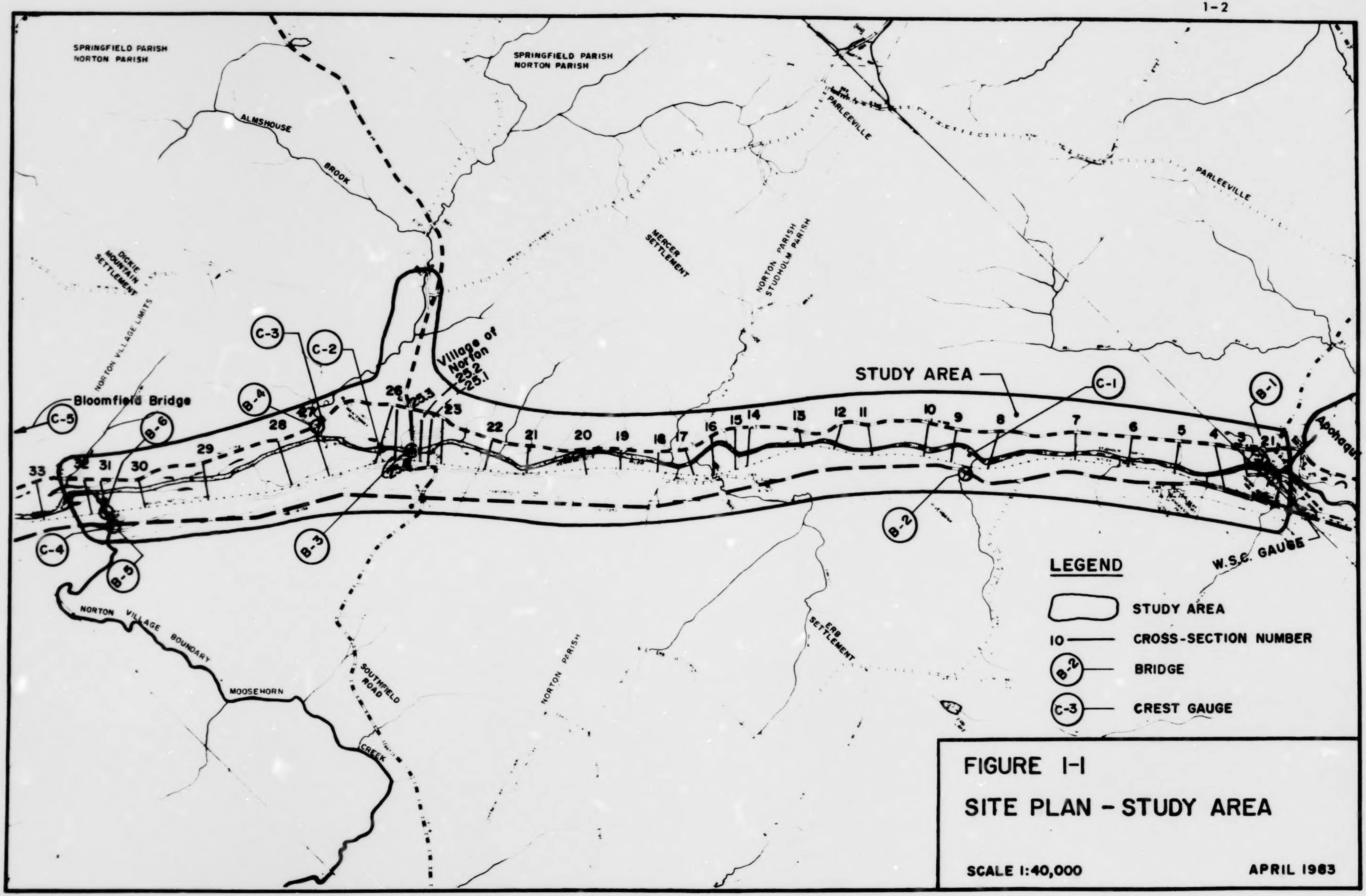

SPRINGFIELD PARISH
NORTON PARISH
SPRINGFIELD PARISH
NORTON PARISH
ALMSHOUSE
BROOK
PARLEEVILLE
PARLEEVILLE
DICKIE MOUNTAIN SETTLEMENT
MERCER SETTLEMENT
NORTON PARISH
STUDHOLM PARISH
NORTON VILLAGE LIMITS
Village of Norton
25.2
25.1
25.3
STUDY AREA
Bloomfield Bridge
C-5
C-3
C-2
C-1
C-4
B-4
B-6
B-3
B-5
B-2
B-1
33
32
31
30
29
28
27
26
23
22
21
20
19
18
17
16
15
14
13
12
11
10
9
8
7
6
5
4
3
21
Apohaqui
W.S.C. GAUGE
NORTON VILLAGE BOUNDARY
MOOSEHORN
CREEK
SOUTHFIELD ROAD
NORTON PARISH
ERB SETTLEMENT
LEGEND
STUDY AREA
10 CROSS-SECTION NUMBER
B-2 BRIDGE
C-3 CREST GAUGE

FIGURE I-I

SITE PLAN - STUDY AREA

SCALE 1:40,000

APRIL 1983

as a result of ice jamming was also addressed in this project using the HEC II ICE option. The results of the Hydrotechnical Study, including the 1:20 and 1:100 year recurrence interval water surface profiles, are given in the main report.

As an intergal part of this study, a programme of field investigations was necessary to obtain the various physical characteristics of the study area. The various facets of the field investigations included:

a) Establishment of vertical control throughout the study area.
b) Cross sections of both channel and flood plain.
c) In-depth study of historic flood levels.
d) Study of ice-jamming problems.
e) Establishing water surface profiles for various flows.

The major portion of the field investigation for this project was carried out by field crews of Fenco Consultants Ltd. and Geocon Inc. during October-December, 1982. During this period all necessary field cross sections, bridge dimensions and elevations of any visible high water ice marks were obtained. Five crest gauges were installed throughout the reach to monitor high flows events. On November 13, 1982 a very high flow in the Kennebecasis River was monitored for the computer model calibration.

Ice jamming problems in the study reach were investigated during a field reconnaissance of March, 1983.

2.0 FIELD SURVEYS - STUDY AREA

2.1 Vertical Control

The proper calibration of the HEC II computer model required that the various field data obtained be referenced to common datum; in this case geodetic. For this purpose, vertical control referenced points or bench marks were established throughout the study reach.

The geodetic bench mark for each field cross section was established along the main CNR track on the south side of the Kennebecasis River. These bench marks, which were usually marked by a spike driven into a railway tie, were established at the southern starting point of each field cross section taken. Bench marks established within the study reach were levelled in from Geodetic Survey of Canada Monuments Nos. 68B194, 77B025, and 12B016. The bench marks for cross sections 1 to 23 were levelled to a closure of 0.004 m (first order) while the closure error for cross sections 26 to 35 was 0.017 m (third order).

Geodetic bench marks were also established at the flow monitoring sites.

2.2 Cross Sections

Calibration of the HEC II computer model requires input of the physical characteristics of the water course within the study area. This information, which was obtained from selected cross sections, made it possible to define both the river channel and flood plain characteristics.

The number and approximate location of the required cross sections was selected by the modelling group at the commencement of the study. The location of each cross section which is shown on Figures 1-1 and 3-1, was based primarily on hydrodynamic principles. Generally cross sections are located at approximately 500-600 m intervals or at any major change in the river course. Cross sections 33, 34, and 35 are located immediately downstream of the confluence of Moosehorn Creek with the Kennebecasis River outside the study area. These sections were included to ascertain the effect of ice jamming in this area on the Norton flooding problem.

All field cross sections within the study reach, with the exception of sections 25.1, 25.2, 25.3, and 25.4, located in the Village of Norton, were obtained in the field using modern surveying techniques. A theotodite T-16 outfitted with an electronic distance measuring device (EDM) was used to measure distances and elevations of the flood plain from the edge of the Kennebecasis River to above elevation +15 m on both north and south sides. Cross sections 25.1, 25.2, and 25.3, and 25.4, were interpolated from New Brunswick Department of Transportation drawings for the recently constructed Norton bridge. In all, a total of 34 cross sections were surveyed.

To define the river channel portion of the cross section, soundings from a 5 m long boat were taken. Originally, it was anticipated that soundings at six to eight locations would be sufficient to accurately define the river channel for backwater modelling. However, after consultation with the Hydrotechnical SubCommittee, it was decided to increase the number of river channel profiles. A total of eleven channel sections, namely cross sections Nos. 1, 3, 7, 10, 13, 17, 20, 23, 28, 31, and 33, were profiled while centre channel soundings were taken at ten additional cross section locations (8, 12, 14, 15, 18, 22, 26, 29, 30, and 32). Channel profile for cross sections at the new Norton bridge were determined from N.B. Department of Transportation drawings.

The New Brunswick grid coordinates for all cross sections within the study area along with their elevations are given in Table 2-1.

The plot of these cross sections and river channel soundings obtained within the study area are shown in Appendix A. These sections have been plotted using the MacLaren CALCOMP plotter. All sections are plotted to scale from left to right looking upstream. In areas where some survey data is missing, such as river channel sections which were not sounded, the computer has interpolated between sections.

TABLE 2-1

NEW BRUNSWICK GRID COORDINATES
CROSS SECTIONS - STUDY AREA

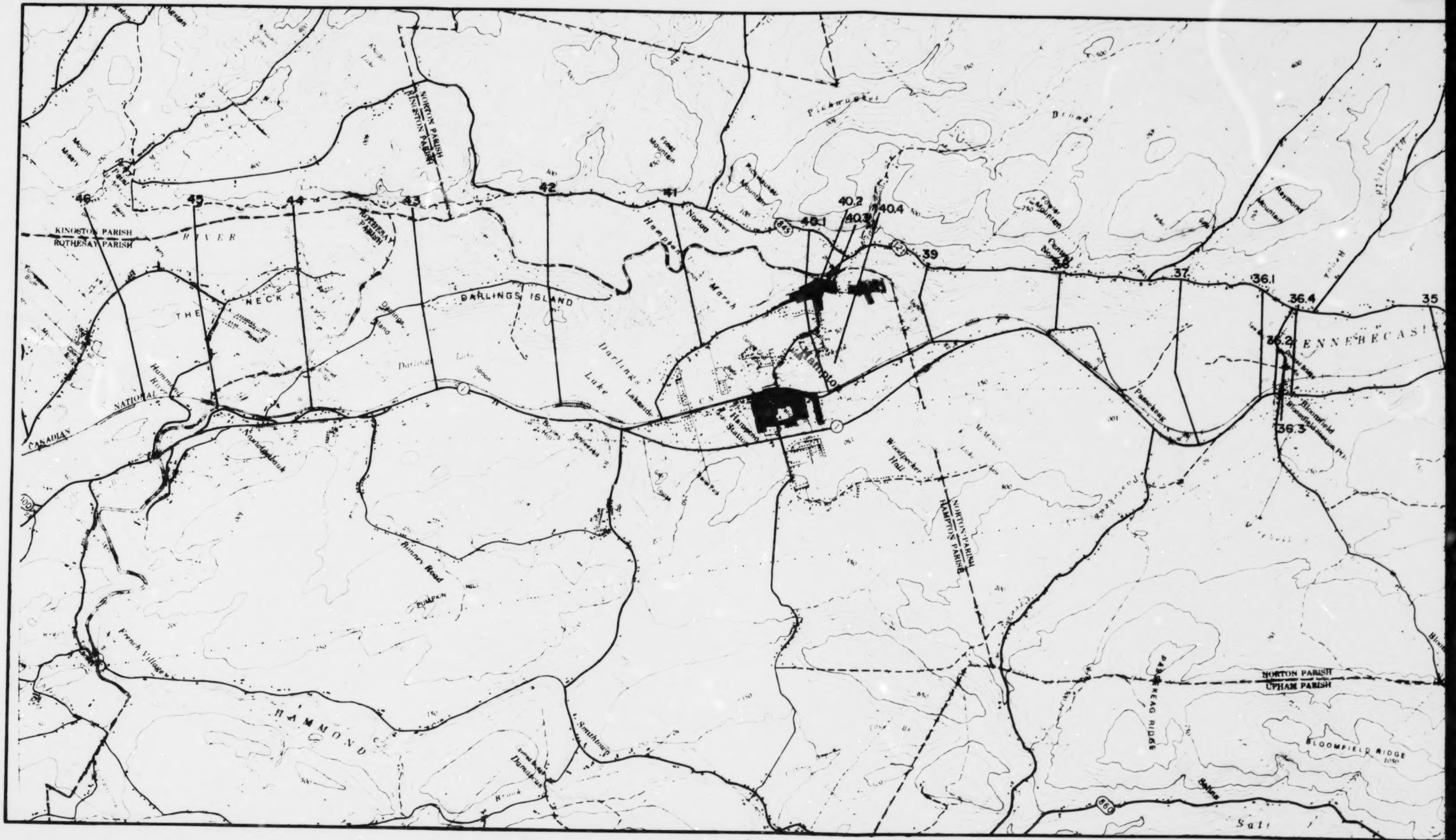
CROSS SECTION NO.	LEFT BANK			RIGHT BANK			NOTES
	N	E	ELEV. (m)	N	E	ELEV. (m)	
1	711 653.2	369 961.6	14.0	711 601.4	369 986.0	14.0	
2	711 731.4	369 908.2	20.7	711 585.2	369 998.2	15.9	
3	711 591.7	369 612.4	19.5	711 395.5	369 829.2	14.1	
4	711 381.2	369 386.0	22.7	711 142.2	369 719.1	20.2	
5	711 107.1	368 975.8	26.3	710 784.0	369 166.9	19.3	
6	710 717.8	368 521.8	39.4	710 422.7	368 709.7	15.6	
7	710 259.4	367 981.4	27.9	710 004.7	368 191.3	16.0	
8	709 637.5	367 376.0	36.7	709 175.2	367 542.8	15.0	
9	708 990.4	366 969.4	42.4	708 990.4	367 264.6	16.7	
10	709 137.4	366 769.5	33.2	708 801.2	366 510.5	18.5	
11	708 571.6	366 221.0	30.9	708 354.6	366 510.5	17.1	
12	708 412.2	366 020.9	26.8	708 107.2	366 186.1	16.0	
13	707 890.7	365 183.6	9.8	707 772.2	365 821.4	14.6	
14	707 576.9	365 307.6	18.1	707 286.6	365 590.3	13.2	
15	707 435.3	365 183.6	14.2	707 095.3	365 506.9	13.7	
16	707 106.9	364 992.5	17.4	706 784.9	365 310.2	13.7	
17	706 835.5	364 908.8	12.1	706 655.3	365 170.8	14.2	
18	706 517.7	364 645.7	16.2	706 320.2	364 772.7	15.0	
19	706 439.7	364 201.8	15.9	706 070.0	364 471.3	15.1	
20	705 926.5	364 010.0	13.8	705 765.9	364 182.8	13.5	
21	705 475.3	363 483.1	14.1	705 234.8	363 729.0	12.7	
22	705 202.5	363 058.3	9.8	704 904.2	363 314.3	16.1	
23	705 027.4	362 604.6	13.4	704 603.0	362 927.2	16.8	
25.1	704 760.4	362 376.4	9.4	704 435.4	362 727.9	16.4	
25.2	704 762.4	362 308.4	10.1	704 389.4	362 677.4	16.2	
25.3	704 762.4	362 308.4	10.1	704 389.4	362 677.4	16.2	
25.4	704 787.4	362 233.4	14.1	704 378.4	362 647.4	15.5	
26	704 666.9	362 023.2	14.6	704 029.8	362 443.3	14.6	
27	703 838.0	361 590.8	8.3	703 534.2	362 111.5	11.5	
28	703 421.9	362 371.3	10.9	703 135.5	361 890.8	10.4	
29	702 255.6	360 929.5	15.8	702 302.8	361 281.1	10.9	
30	701 859.3	360 512.0	13.6	701 658.6	360 590.5	11.3	
31	701 634.5	360 281.5	8.9	701 398.6	360 590.5	11.4	
32	701 281.8	359 912.2	8.5	701 139.9	360 384.6	11.4	
33	701 012.7	359 394.7	14.8	700 758.3	360 090.6	11.5	
34	700 593.4	359 365.0	11.6	700 320.1	359 753.5	10.3	
35	700 113.3	359 190.8	12.8	699 930.5	359 503.9	9.4	

3.0 DOWNSTREAM HYDRAULIC MODEL

As indicated in the Terms of Reference, in order to delineate the flood hazard within the study area, it would be necessary to determine what, if any, tidal backwater effect there is within the study area. To investigate this effect, a very course HEC II backwater model of the Kennebecasis River between Oliver Point and the downstream limit of the study area was utilized. This preliminary model was calibrated using cross-sections obtained from both field surveys and 1:10 000 orthophoto mapping. The location of these cross sections are shown on Figure 3-1.

The field cross sections for the downstream model were obtained at the Hampton and Bloomfield bridges. At these bridges, the Kennebecasis River's channel bottom was sounded from the bridge deck and referenced to geodetic elevation. The representative downstream cross sections obtained from the 1:10 000 mapping are spaced at approximately 1 600 metres apart.

A total of twenty cross sections, numbered from 33 to 46, were used in this preliminary model. The computer plot of each of these sections is shown in Appendix B. On the plots of cross sections 36.3 and 40.3, located along the centrelines of the bridges at Bloomfield and Hampton respectively, the restrictions to flow caused by these bridges are indicated.


46
45
44
43
42
41
40.1
40.2
40.3
40.4
39
37
36.1
36.4
35
36.2
36.3
KINGSTON PARISH
ROTHESAY PARISH
NORTON PARISH
KINGSTON PARISH
ROTHESAY PARISH
NORTON PARISH
HAMPTON PARISH
NORTON PARISH
UPHAM PARISH
RIVER
THE NECK
DARLINGS ISLAND
KENNEBECASIS
CANADIAN NATIONAL
Hampton
Hampton Marsh
Darlings Lake
Lakeside
Hammond River
Nauwigewauk
Mount Misery
Frost Mountain
Pickwauket Mountain
Pickwauket
Flower Mountain
Raymond Mountain
Central Norton
Bloomfield
Bloomfield Station
Passekeag
Woodpecker Hall
Bunney Road
PUMPKIN HILL
French Village
HAMMOND
Smithtown
Damascus
PASSEKEAG RIDGE
BLOOMFIELD RIDGE
Salt
Spoon
Broad
845
121
100
860
1

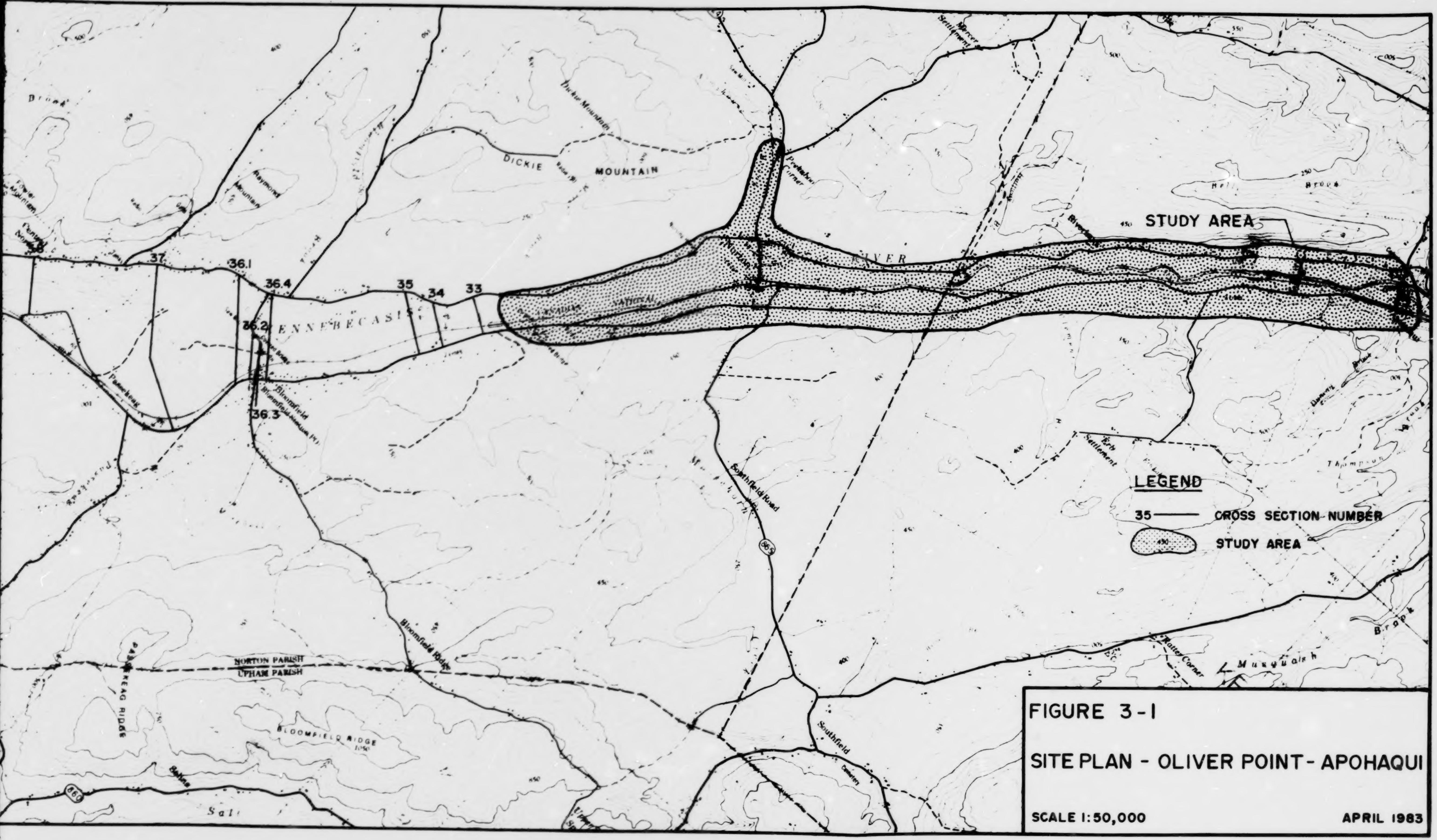
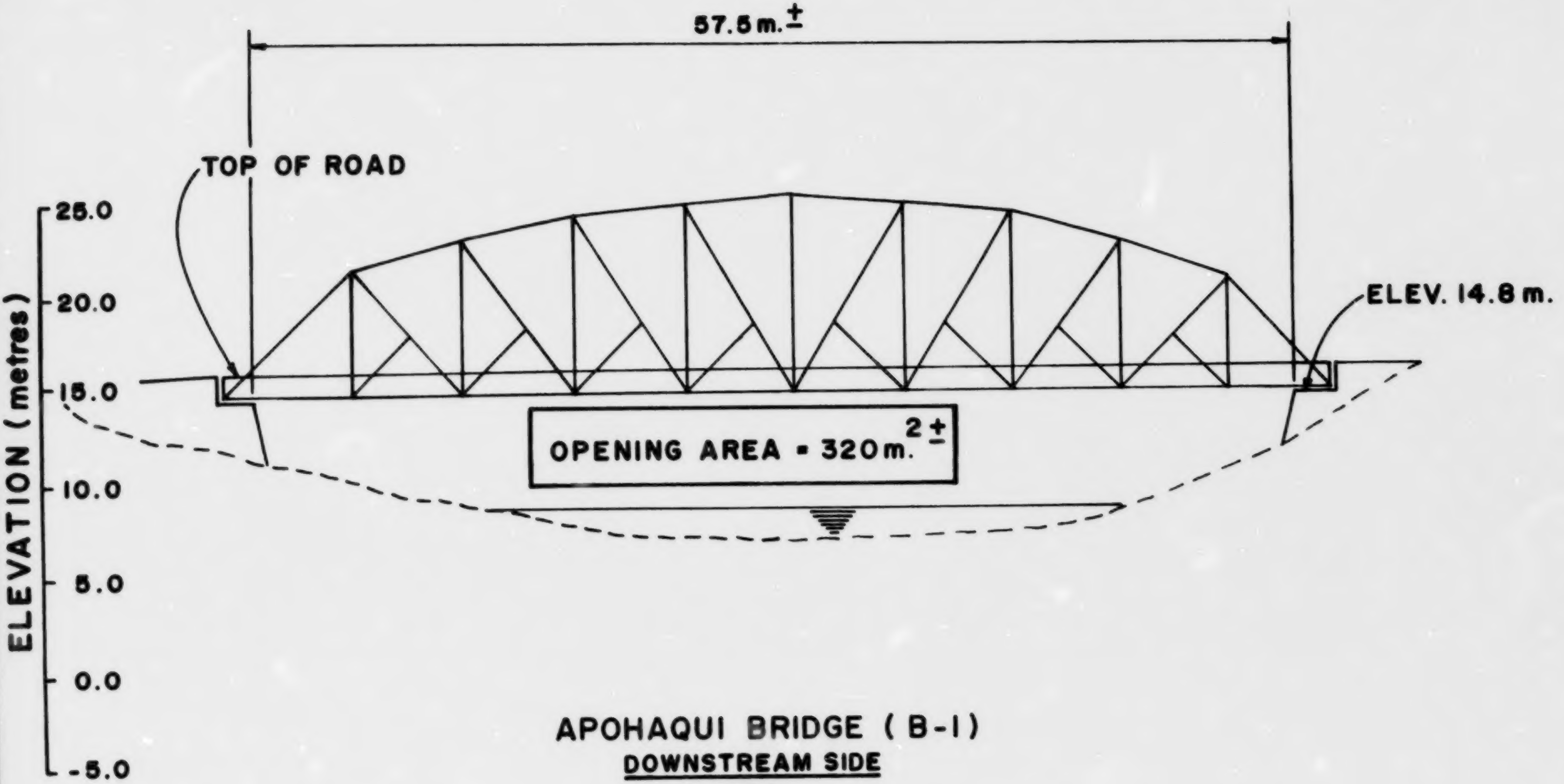

STUDY AREA
37
36.1
36.4
36.2
36.3
35
34
33
DICKIE MOUNTAIN
Dickie Mountain
Raymond Mountain
KENNEBECASIS
RIVER
CANADIAN NATIONAL
Norton
Passekeag
Bloomfield
Southfield Road
865
Bloomfield Ridge
NORTON PARISH
UPHAM PARISH
BLOOMFIELD RIDGE
1050
Southfield
Mercer Settlement
Erb Settlement
Hell Brook
Thompson Brook
Musquash
860
LEGEND
35 CROSS SECTION NUMBER
STUDY AREA

FIGURE 3-1

SITE PLAN - OLIVER POINT - APOHAQUI

SCALE 1:50,000

APRIL 1983


57.5 m. ±
TOP OF ROAD
ELEV. 14.8 m.
OPENING AREA = 320 m.^{2} ±
ELEVATION (metres)
25.0
20.0
15.0
10.0
5.0
0.0
-5.0

APOHAQUI BRIDGE (B-1)
DOWNSTREAM SIDE
N.T.S.

FIGURE 4-1

BRIDGE DESCRIPTION

NORTON HYDROTECHNICAL STUDY

Bridge: CNR Bridge Mile 50.1 (B-2)

Date Constructed: -

Water Course: Secord Brook

Type: Standard steel tresle

Number of Spans: One

Pier Description: -

Length of Span: 8.5m

Deck Elevation: -

Bottom Chord Elevation:

Opening Size: 50m^2 (approx.)

Channel Characteristics: -

Notes: In 1981, ice jam between railway and highway No. 1 caused Secord Brook to backup. When jam broke, resulting flows underscoured west abutment. The foundation of this abutment was repaired in 1981.

BRIDGE DESCRIPTION
NORTON HYDROTECHNICAL STUDY

Bridge: Norton Bridge (B-3)

Date Constructed: 1981

Water Course: Kennebecasis River

Type: Steel Truss

Number of Spans: One

Pier Description: -

Length of Span: 60m±

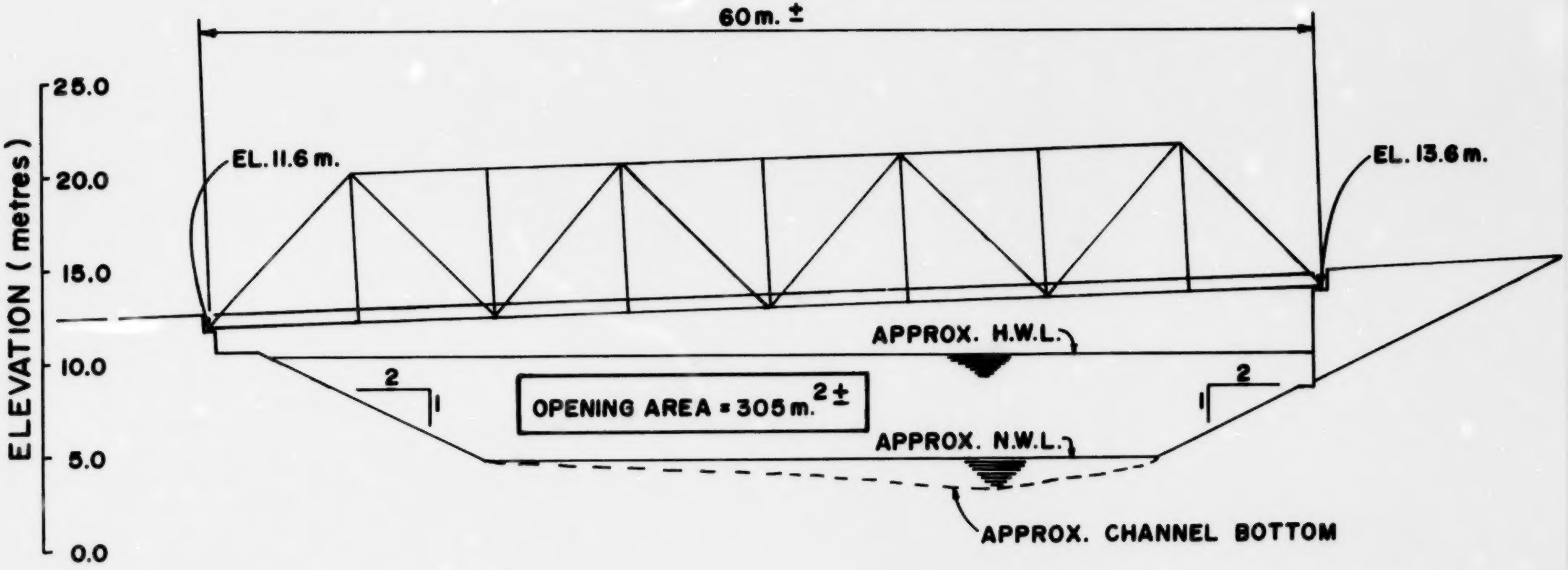
Deck Elevation: -

Bottom Chord Elevation: 11.6m, 13.6m

Opening Size: 305m^{2}±

Channel Characteristics: Channel skewed south

Notes: Bridge constructed in 1981 to replace structurally unsound existing structure.


60m. ±
25.0
20.0
15.0
10.0
5.0
0.0
ELEVATION (metres)
EL. 11.6 m.
EL. 13.6 m.
APPROX. H.W.L.
APPROX. N.W.L.
OPENING AREA = 305 m.2 ±
2
1
2
1
APPROX. CHANNEL BOTTOM

NORTON BRIDGE (B-2)
DOWNSTREAM SIDE
N.T.S.
FIGURE 4-2

BRIDGE DESCRIPTION
NORTON HYDROTECHNICAL STUDY

Bridge : Highway No. 121 Bridge (B-4) Date Constructed : -

Water Course: Almshouse Brook

Type : Reinforced Concrete Number of Spans: One

Pier Description : - Length of Span : 6.7m

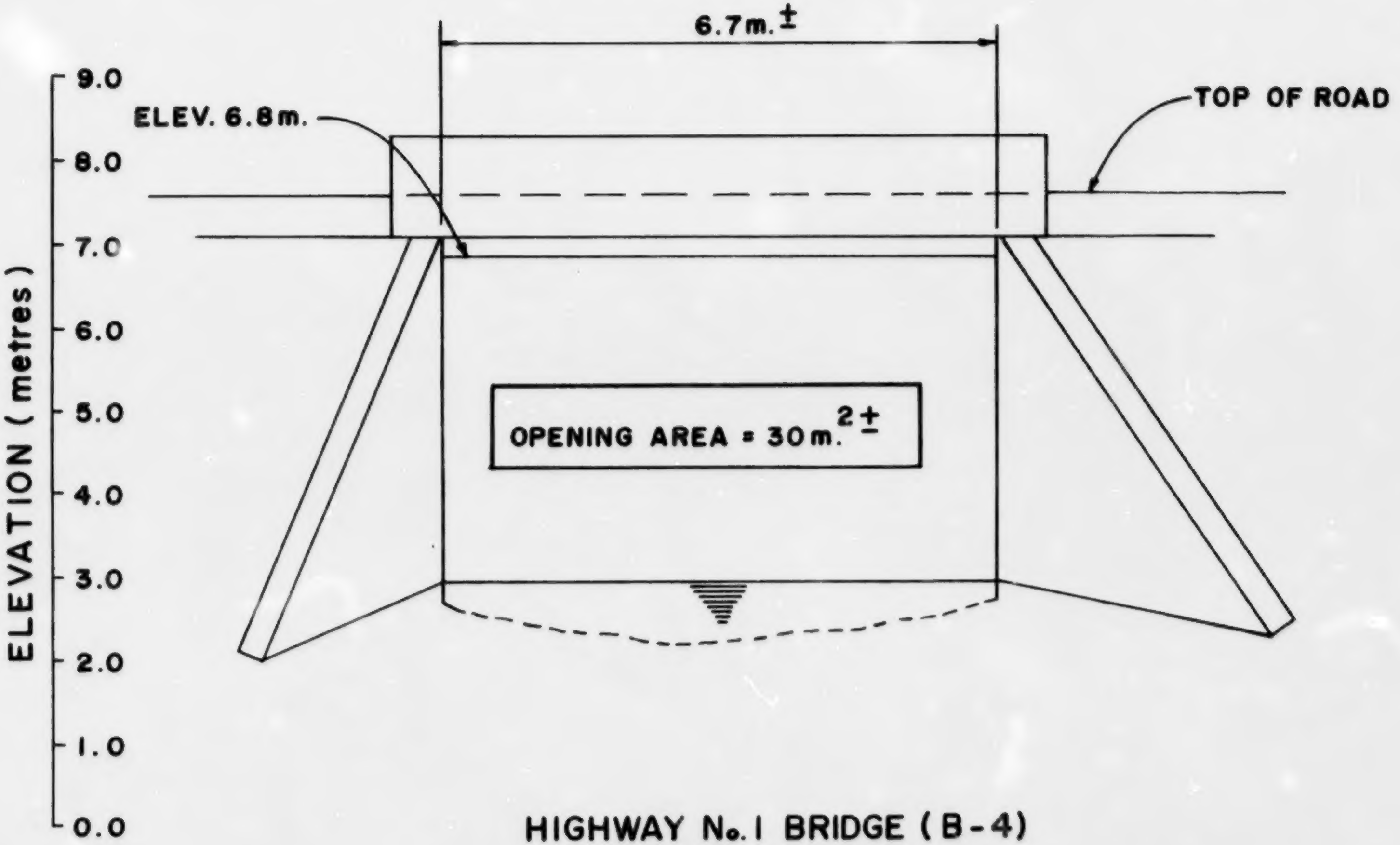
Elevation (Road): 7.6m

Bottom Chord Elevation: 6.8m

Opening Size: 30m^{2}±

Channel Characteristics: -

Notes: Crest gauge attached on upstream side of bridge.


6.7m.±
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
ELEVATION (metres)
ELEV. 6.8m.
TOP OF ROAD
OPENING AREA = 30 m.^{2}±

HIGHWAY No. I BRIDGE (B-4)
ALMSHOUSE BROOK
UPSTREAM SIDE
N.T.S.

FIGURE 4-3

BRIDGE DESCRIPTION
NORTON HYDROTECHNICAL STUDY

Bridge: Highway No. 1 (B-5)

Date Constructed: 1962

Water Course: Moosehorn Creek

Type: Reinforced Concrete

Number of Spans: One

Pier Description: -

Length of Span: 17.5m

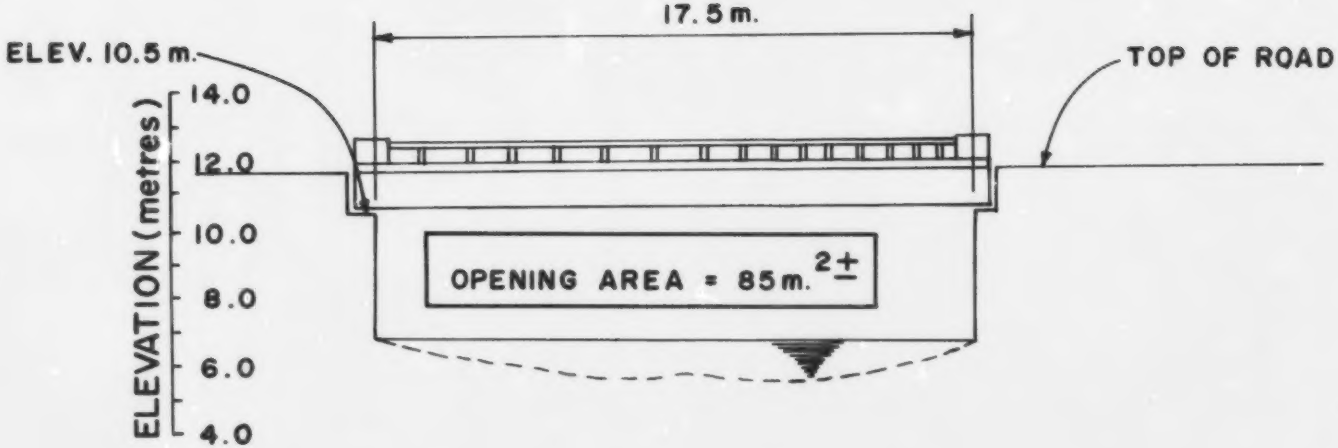
Deck Elevation: 11.5m

Bottom Chord Elevation: 10.5m

Opening Size: $85m^{2\pm}$

Channel Characteristics: Gravel Covered

Notes: Crest gauge attached to west abutment on D/S side.


17.5 m.
ELEV. 10.5 m.
TOP OF ROAD
ELEVATION (metres)
14.0
12.0
10.0
8.0
6.0
4.0
OPENING AREA = 85 m.^{2}±

HIGHWAY No. 1 BRIDGE (B-5)
MOOSEHORN CREEK
DOWNSTREAM SIDE
N.T.S.

FIGURE 4-4

BRIDGE DESCRIPTION
NORTON HYDROTECHNICAL STUDY

Bridge : CNR Bridge Mile 56.5 (B-6)

Date Constructed : -

Water Course: Moosehorn Creek

Type : Through Plate Girder

Number of Spans: One

Pier Description: Block Construction

Length of Span : 23.9m

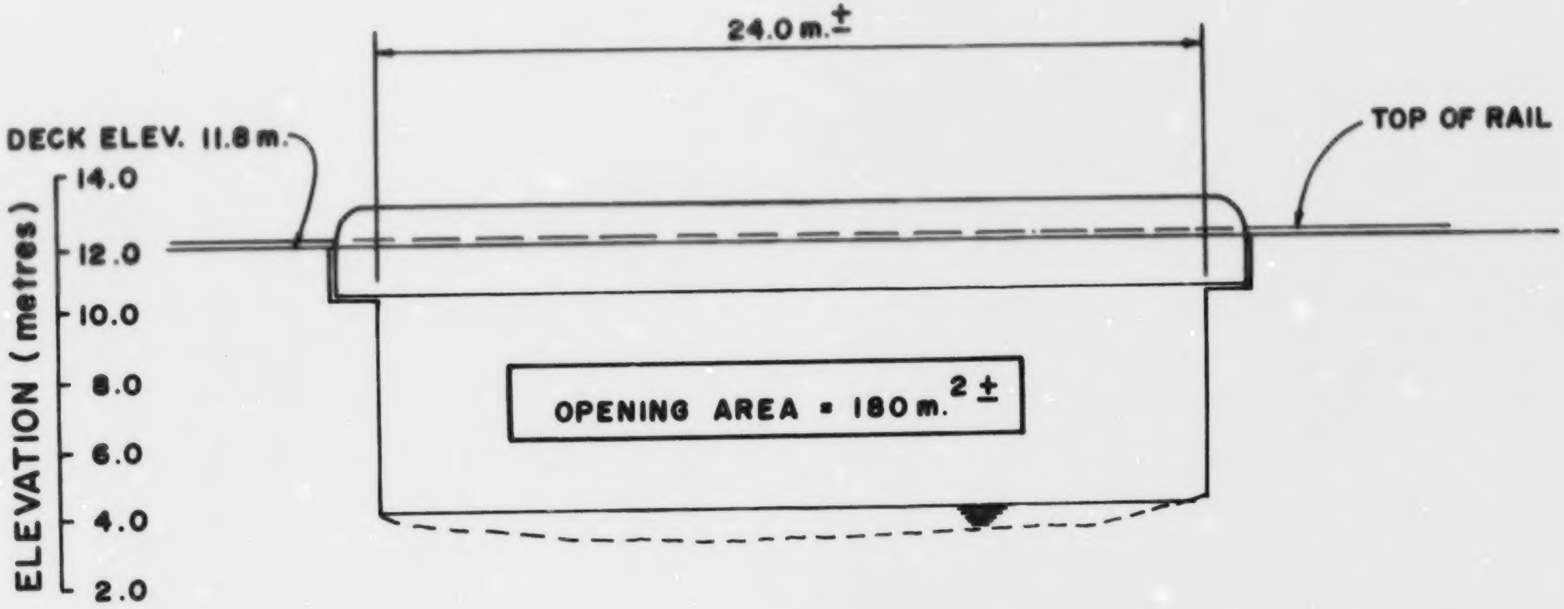
Deck Elevation 11.6m

Bottom Chord Elevation : -

Opening Size : $160m^2$ ±

Channel Characteristics: Gravel covered

Notes : Channel has a 75° bend immediately upstream of bridge. Current is causing scour under west foundation.


24.0 m.±
DECK ELEV. 11.8 m.
TOP OF RAIL
ELEVATION (metres)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
OPENING AREA = 180 m.^{2} ±

C.N.R. BRIDGE - MILE 56.5 (B-6)
MOOSEHORN CREEK
DOWNSTREAM SIDE
N.T.S.

FIGURE 4-5

5.0 HISTORIC FLOOD REVIEW

5.1 General

An intergral component of this flood study was the accumulation and assessment of all historical flood data for the Norton area. Included within this phase of the project was the dating of all previous floods in the area, determining the causes, and assessing the severity of the flood.

A number of sources were employed in the gathering of this information. The most important sources from which information was gathered were the reports by M. E. Folster, and the New Brunswick Department of the Environment, Inland Waters Directorate Atlantic Region summarizing regional historical flooding information from various sources for this region. In addition, information gathered from interviews with local residents, particularly Mrs. Lil Reid, a local historian, was used to expand information obtained from reports.

Table 5-1 summarizes the information gathered regarding dates, causes, and effects of the major recorded floods in the Norton area from 1854 to 1981.

TABLE 5-1

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
1854 Nov 12-14	Rain	Water rose rapidly from Sussex to Hammond River. Cattle drowned in their stalls.
1887 -	-	No data available although event is mentioned in reports from later years.
1902 Mar 17-23	High temperature and heavy rain. Ice jams.	At Lower Millstream considerable damage occurred when mill dam burst. Highways and bridges in the vicinity of Norton were damaged or swept away. Norton conditions were worst since 1854.
1909 Jan 04-07	Mild weather and heavy rains. Possible ice jams.	14 bridges destroyed in Kings Co. between Sussex and Saint John. At Norton, flooding resulted in the evacuation of several barns. The only method of conveyance was by boat.
1923 Apr 09-12	Snowmelt and ice jams - 3 days of mild weather.	At Apohaqui, cellars flooded. On April 10 the ice broke at Sussex and jammed at Norton. By 9 o'clock of that day, water was flowing over the road. Water rose over road inundating houses and shops, reaching 2 feet (0.6 m) in some cases. Jam broke later that day and water receded quickly. Freshet described as "highest in years."
1923 Apr 29-May 09	Snowmelt, heavy rain, warm temperatures.	At Norton several buildings were threatened but water receded before entering structures. Water to within 6 inches (0.15 m) of B. Perley Wood's drug store floor. At Bloomfield all interval land inundated, 200-300 feet (60-90 m) of road between the bridges submerged. The Kennebecasis River was said to be higher than it was in the memorable floods of 1887 and this was the second flood this spring.

TABLE 5-1 (Cont'd.)

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
1936 Mar 16-25	High temperatures for 2 days. Ice jams.	Flooded areas at Norton, Hampton, Rothesay. Flooding extensive at Norton forcing people to evacuate. Flood waters highest in Norton. Ice over road at Wilcox Brook. Road grade is now 3 feet (0.9 m) higher. Families had to be evacuated from second storey windows.
1945 Feb 05-06	Rain, mild temperatures, ice jams.	At Norton two ice jams brought flood water into several homes and stores near the river. One jam located at the Highway bridge while the other formed about 2 miles (3.2 km) D/S of community. The lower jam was reported to have broken under the pressure before the situation became too serious.
1947 Feb 05	Torrential rains, high temperatures, ice jams.	Ice jams at Norton. Worst flood in at least 6 years. Ice jammed at Norton Bridge and 2 miles (3.2 km) further down river from Norton Bridge. Water in homes and stores.
1950 Nov	Intense rainfall.	Worst fall floods in Kings County since 1938.
1951 Feb 07-08	Mild temperatures, rain, melting snow, ice jams.	At Norton highways were under water at some points and the street leading into Norton covered by more than 1 foot (0.3 m) of water. Other streets in the community were also under water and several homes surrounded. In some instances houses had water on the ground floor. Water approached within 2 feet (0.6 m) of Highway No. 2. Flooding was the result of ice jam 2 miles (3.2 km) downstream.

TABLE 5-1 (Cont'd.)

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
1952 Jan 24-25	Rain, mild temperatures, ice jam.	Recent rains have raised the Kennebecasis to almost flood level several times, but this storm resulted in a heavy ice jam midway between Norton and Bloomfield. At Norton water levels reached "dangerous" levels. Fields were flooded and about "a one-foot (0.3 m) rise would have sent water over the main street in the vicinity of Norton Superior School". Some basement flooding occurred.
1953 Feb 07-08	Torrential rains, mild temperatures, ice jam. 2.70 in (69 mm) of rain at Sussex in less than 24 hours.	In Norton Area an ice jam 2 miles (3.2 km) west of Norton backed up water to the danger point before breaking under its own pressures. Fences were broken and other minor damage was reported to be caused. The water level in the Kennebecasis River was estimated to be 20 feet (6 m) above normal summer levels before the ice jam broke at mid-day February 8th.
1957 Jan 22-23	Mild temperatures, rain and melting snow.	A danger point in the Kennebecasis River, 2 miles (3.2 km) west of Norton, was reported "to be free of ice jams".
1961 May 25-31	Severe extra-tropical storm.	In Kings County, rivers and streams overflowed their banks but no serious damage was reported other than some minor washouts. CPR cancelled trains one being Chipman to Norton. The streams in Kings County were reported to have been higher than the annual spring freshet which was below normal this year.
1962 Mar 31-Apr 04	Heavy rains. 5.04 in (128 mm) at Sussex	At Apohaqui 40,000 board feet (95 m^3) of logs at Jones Bros. Limited sawmill were washed away. At Norton, river inundated stores, homes and cut off the southern section of the community from outside communication. The main street was under 3 feet (0.9 m) of water for several hours.

TABLE 5-1 (Cont'd.)

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
		The Village of Norton suffered the most in terms of flood damage and the storm was described as the worst in the area since the April-May flood of 1923 which was "the greatest freshet since 1887" (Kings County Record May 11, 1923). Two stores, E.K. Cripps and Perkins Hardware were flooded to a depth of 2 feet (0.6 m) or more. The water was reported to have reached the step below the cement entrance to Burley's Mart. Many homes in Norton could be reached only by boat. No ice jam.
1970 Feb 02-06	Heavy rain, high temperatures and ice jams. More than 4 in (200 mm) of rain.	In Kings County heavy damage was reported to have occured to secondary roads and several were closed because of washout. At Norton water was reported to be lapping against some low lying buildings in the village.
1973 May 03	Spring thaw.	N.B. Environment photo shows road at Bloomfield Station under water.
1976 Jan 27-Feb 02	Persistent rain. Mild temperature.	Flooding general in watershed although no direct reports for Norton.
1979 Mar 15	Heavy rain. Snowmelt.	Flooding of low lying areas. N.B. Environment photos show water up to low point on road through Norton and touching back of out-buildings. Also close to road level at Bloomfield Station.

TABLE 5-1 (Cont'd.)

HISTORICAL REVIEW OF FLOODING IN NORTON AREA

Date	Cause	Notes
1981 Feb 11-13	Rain, mild temperatures, melting snow and ice accompanied by high winds. Some ice jams were also reported but probably not at time of peak flow.	Flooding in low lying areas was reported throughout Kings County. A train bridge abutment at Wilson Brook between Apohaqui and Norton was washed out. At Norton the main road was out and about 12 properties were surrounded by flood waters. The water level at the Apohaqui gauge peaked at a record high. A mile of underground cable was also washed out cutting service to the New Brunswick Telephone customers in the Springfield-Norton area. Extent of flooding well documented by areal photos taken by New Brunswick Environment.

5.2 High Water and Ice Marks

During the course of the field investigations, geodetic elevations were established on any historic high water mark which could be identified throughout the study reach. The main source of this historic information was copies of photographs taken during previous flooding events in the area. These pictures were supplied to us by the New Brunswick Department of the Environment. Information on historic high water elevations was also obtained from N.B. Department of Transportation drawings and interviews with local residents. A summary of identified high water elevations is given in Table 5-2 while Figures 5-1 and 5-2 graphically illustrate the extent of previous flooding events.

Ice scars are evident on the bark of many trees throughout the study area. The scars are observed on trees lining the water course as well as on those situated well back on the flood plain. During field surveys, elevations were established for many observed scars. A summary of the location and elevation of these ice scars is given in Table 5-3 while Figures 5-1 and 5-2 illustrate graphically their location.

TABLE 5-2

ELEVATION OF HIGH WATER MARKS - STUDY AREA

LOCATION	SECTION NO.	DATE	HIGH WATER ELEVATION (m)	NOTES
Norton	25.3	1939	8.5	Photo of Main Street.
Norton	25.2	1962	9.6	Photo of Langell house.
Norton	25.2	1962	9.1	Photo of Mae Northrup house.
Norton	25.3	1962	10.0	
Norton	25.3	1962	9.3	Photo of E.K. Cripp house.
Norton	25.3	-	10.2	N.B. D.O.T. plan of new Norton Bridge.
Norton	25.3	-	8.9	Saunders house. Highest flood since 1942 (personal communication).
Norton	25.3	1981	8.7	Aerial photo of flood near new bridge.
Norton	25.3	1981	8.4	Aerial photo of flood near N.B. Tel Building.

TABLE 5-3

RECORDED ICE MARK ELEVATIONS - STUDY AREA

SECTION NO.	ICE MARK ELEVATION (m)
7	12.6
10	11.7
11	11.6
13	11.6
14	11.9
16	13.8
17	11.4
19	9.5
19	10.0
20	9.1
22	9.3
26	8.2
27	7.8
29	7.3
30	6.9
30	7.0
31	7.3
31	7.4
32	6.7
33	6.6
34	6.4
34	7.0

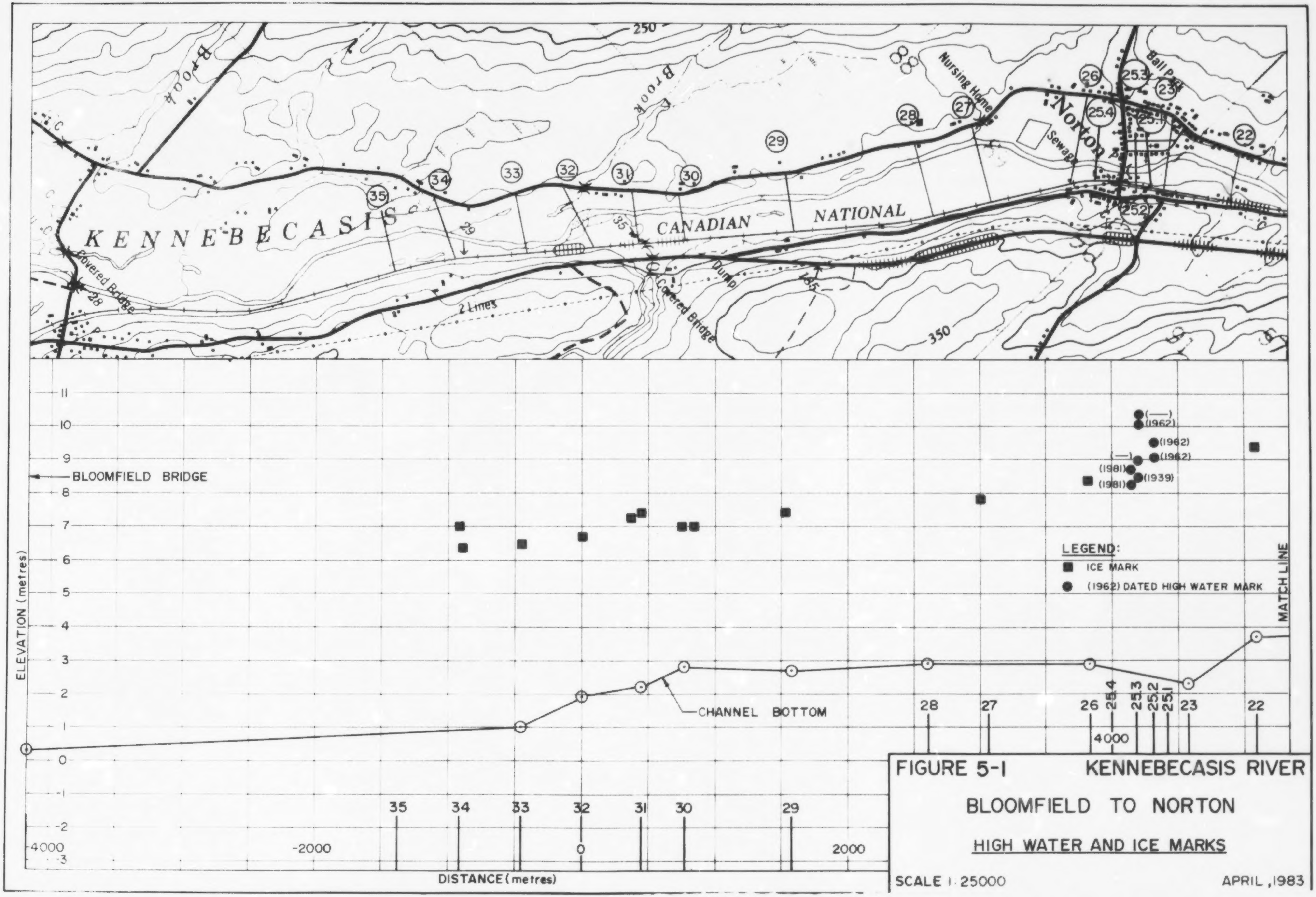

KENNEBECASIS
CANADIAN NATIONAL
Norton
Nursing Home
Ball Park
Sewage
Brook
Covered Bridge
Dump
2 Lines
250
350
35
34
33
32
31
30
29
28
27
26
25.4
25.3
25.2
25.1
23
22
BLOOMFIELD BRIDGE
CHANNEL BOTTOM
ELEVATION (metres)
DISTANCE (metres)
-4000
-2000
0
2000
4000
(—)
(1962)
(1962)
(1962)
(—)
(1981)
(1939)
(1981)
LEGEND:
■ ICE MARK
● (1962) DATED HIGH WATER MARK
MATCH LINE

FIGURE 5-1 KENNEBECASIS RIVER

BLOOMFIELD TO NORTON

HIGH WATER AND ICE MARKS

SCALE 1:25000 APRIL, 1983

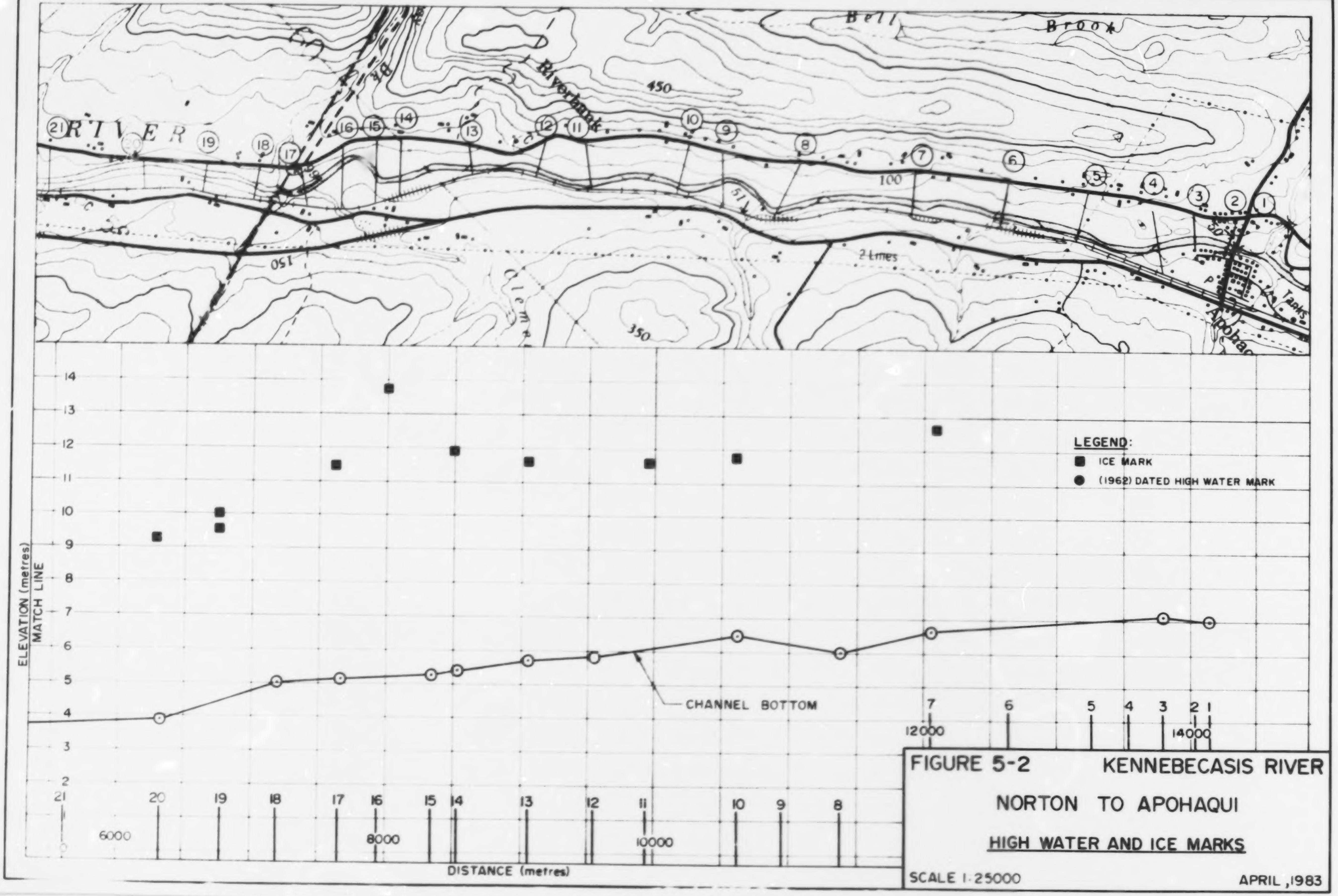


FIGURE 5-2 KENNEBECASIS RIVER

NORTON TO APOHAQUI

HIGH WATER AND ICE MARKS

SCALE 1:25000

APRIL, 1983

6.0 ICE OBSERVATION PROGRAMME

6.1 Introduction

A hydrotechnical field reconnaissance was conducted on 30 March 1983, as a component of our Ice Studies. As the ice run was a minor event, the date was selected to coincide with a period of relatively high flow, and the type of hydrodynamic conditions which would represent a spring condition in the river.

The river and its tributaries were initially inspected from Sussex to Hampton, to obtain an appreciation of the effects which could be induced in the study reach by downstream and upstream ice action. Subsequent detailed inspection concentrated on the reach between Mercer Brook and Mill Stream, and focused on known or anticipated ice jam initiation sites within this reach.

6.2 Field Observations

Above Study Reach East of Apohaqui

There are a number of bends, narrows, shallows and tributary confluences above the study area, and their configurations suggests that frequent (perhaps temporary) ice jams occur here. The possibility of a surge of ice and flood wave from these locals moving downstream into the study reach is quite likely and may cause premature or dry ice jams in the study reach.

Below Moosehorn/Mercer Brook Near Bloomfield

Numerous meanders and the reverse bend below Bloomfield (Cross Section 36) will obstruct clearing of ice from the Kennebecasis River and contribute to regular flooding below Bloomfield. However, the wide overbanks below Bloomfield (Cross Sections 37-39) and conveyance in these overbanks should dampen backwater ice effects upstream of Bloomfield. There appears to be some ice jamming potential at Bloomfield Bridge, or at the river curves about 1-2 km upstream. The absence of physical evidence of frequent high water levels, however, suggests that the project study reach is practically independent of ice effects in the area below it.

Channel Section No. 32, Moosehorn/Mercer Brook Confluence

The bend, islands and relief channels in this area were carefully inspected on foot, and photographed for reference from both sides of the river. The reach presents a location where a consolidated ice cover would have sufficient strength to resist even high flows. A cover at this location would likely remain in place and contribute to initiating ice jams on a frequent basis, interrupted only by meteorological conditions which would cause it to weaken and decay before the spring freshete. Moreover, the bend and bars here would contribute to making this ice highly resistent to even a surge of ice and water released by the break-up of upstream jams. As an aside, the potential for jams at this site may be further (but slightly) augmented by the influx of frazil or break-up from Moosehorn Creek and Mercer Brook - but for the most part, ice jams at this site are expected to be deep and intiated on the northwest bank at this bend.

Of all the possible ice jam sites observed along the Kennebecasis River within the study reach, this site represents perhaps the greatest potential for frequent and significant jamming, and upstream backwater conditions will be investigated using the Model HEC-2-ICE.

Area of Almshouse Brook

There is a shallow bar on the east bank of the Kennebecasis River just upstream of the confluence of Almshouse Brook, which might initiate a jam and ice accumulation up through Norton. This reach was photographed and noted for future study.

Norton Railroad Bridge

The reach between the road and rail bridges at Norton was examined in detail and photographed. The channel bends at a bottle-neck below the road bridge and then carries through the remains of the former railroad bridge. Although the deck and a portion of the central pier of the railroad bridge have been removed, the remaining abutments and central pier still form points on which an ice arch could develop (particularly if a downstram jam forms through this reach or a large flux of ice enters the area from the release of an upstream jam).

Although the hydraulic characteristics of this location suggests that ice jamming would be less frequent here than near Moosehorn Creek, the backwater effects could be significant at Norton, and consequently this reach should also be modelled (HEC-2-ICE).

Riverbank Near Halfway Brook and Clements Brook

This area, near Station 15 and 16, was examined in some detail as relatively sharp bends would present an obstruction to the clear passage of ice and provide protection for the consolidated ice sheet on the western bend (Section 16). Logs and other debris are lodged on the western bank and indicate that high flows would force ice onto this bank as well.

The overbank area here provides significant release capacity for ice jams occurring at Section 16, and backwater effects would not be anticipated to be large. The same effect is present at Section 15, but because it presents an ideal location for jamming, it would be an appropriate site to model (analytically).

Secord Brook - Section 8-9

This location was also examined in detail because it combines a bend, a shoal, a frazil and break-up ice source (Secord brook) and an anchor ice location all at one site. These factors should combine frequently to initiate an ice jam at this site and it will be appropriate to analyse the hydraulics under open water conditions to determine the type and stability of possible ice jams at this site.

6.3 Summary and Conclusions

The reconnaissance was of considerable value as it provided an opportunity to select and examine potential ice jam sites during representative spring conditions.

Conditions near the mouth of Moosehorn Creek and near the rail crossing at Norton stand out as sites which would frequently be the initiation point for ice jamming and which should be simulated by HEC-ICE.

The bend near Riverbank and at Secord Brook are also quite likely to obstruct the downstream movement of ice - particularly if the break-up of ice jams above Apohaqui rapidly introduce a large influx of ice through the study reach. Backwater conditions above these sites will be examined using analytical tools, and in all likelihood, the Secord Brook will be assessed numerically.

Several other locations, such as just upstream of Almshouse Brook, or the shoal at Section 7, were identified in the field as potential contributors to ice problems in the study reach. It will be appropriate to examine the progression of break-up through these reaches using the results of HEC-2 open water simulations, and including these in HEC-ICE simulations.

7.0 WATER SURFACE PROFILES

7.1 Water Level Measurements

Calibration of the HEC II computer model requires several water surface profiles along with flows associated with these levels. In the Terms of Reference, it was expected that 2-3 events would be recorded during the study period for this calibration.

Discharge measurements for the Kennebecasis River were obtained by prorating the recorded water levels at the Water Survey of Canada gauge at Apohaqui. Initially, it was felt that it would also be necessary to measure the discharge of the Almshouse Brook tributary. However, after careful study, it was decided this monitoring was not necessary for the following reasons;

1. It appears that Almshouse Brook peaks approximately twelve hours before the Kennebecasis River, and

2. Initial estimates of the 1:100 year flood indicate that backwater from the Kennebecasis River will dominate this brook.

As an aid to measuring peak flow water levels, a number of crest gauges were installed throughout the study area in the latter part of November, 1982. These crest gauges, which were intended to monitor the high flows expected during the spring of 1983, were installed either at bridge locations or at easily accessible locations throughout the study area. In addition, a crest gauge was installed at the Bloomfield bridge, outside the study area. The location description of each crest gauge is given in Table 7-1 while exact location has been indicated on Figure 1-1. Sketches of each of the crest gauge locations are shown in the following figures 7-1 to 7-5.

TABLE 7-1

CREST GAUGE LOCATION

CREST GAUGE NO.	RIVER COURSE	LOCATION	DESCRIPTION
C-1	Kennebecasis River	Section No. 9	Mounted to a large elm tree about 15 m from south river bank.
C-2	" "	Norton	Mounted to the old railway bridge abutment north side of river, Section #26.
C-3	" "	Bloomfield Bridge	Mounted to a large maple tree 15 m D/S of bridge on north side.
C-4	Almshouse Brook	Hwy. No. 121 Bridge	Mounted on U/S side of bridge east abutment.
C-5	Moosehorn Creek	Hwy. No. 2 Bridge	Mounted on D/S side of bridge on east abutment.

During the period October 26, 1982 to April 26, 1983, there were several large flows measured in the Kennebecasis River. These

Crest Gauge C - 1
- mounted on large elm tree 15 meters from river bank

Bench Mark
- spike driven in railway tie at section 9 elevation +16.724 m

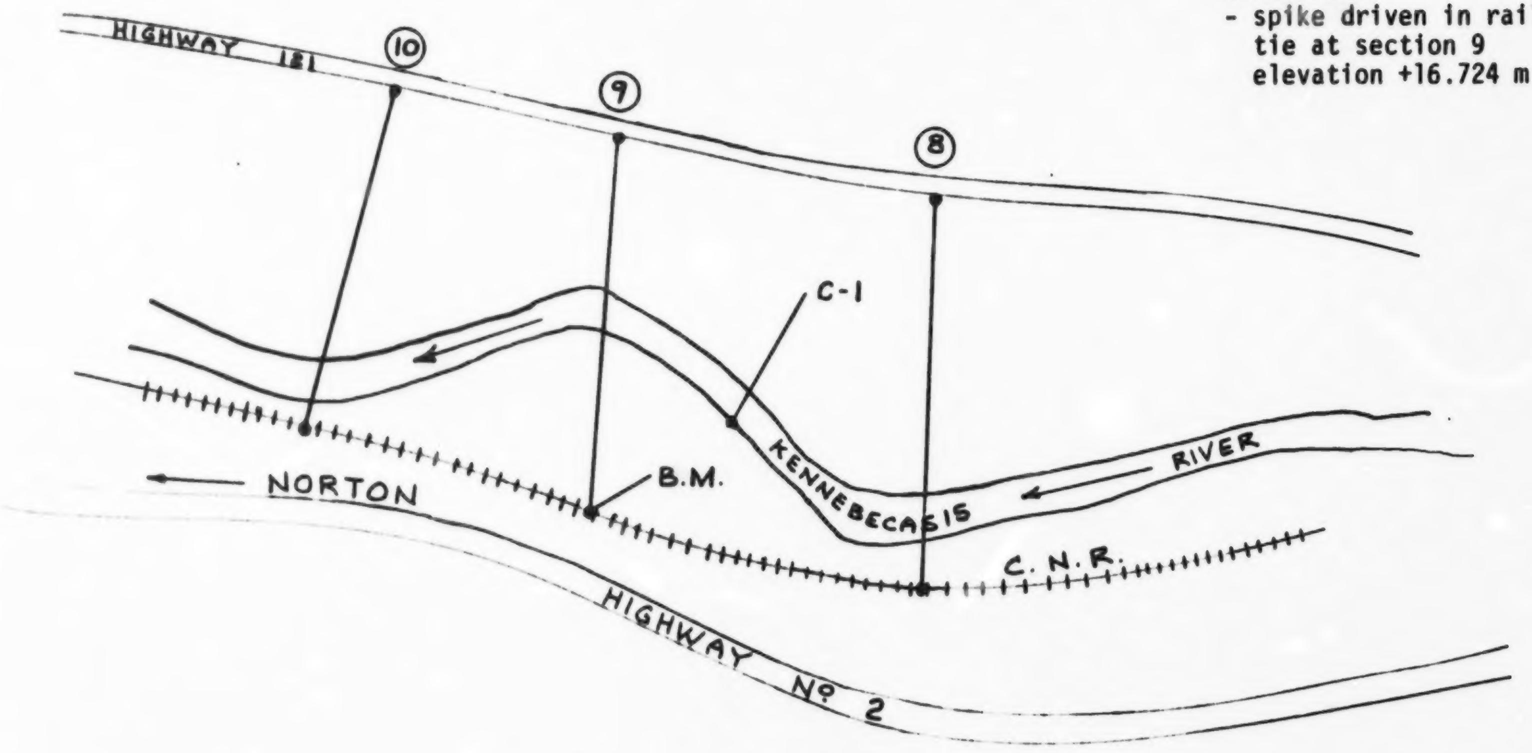
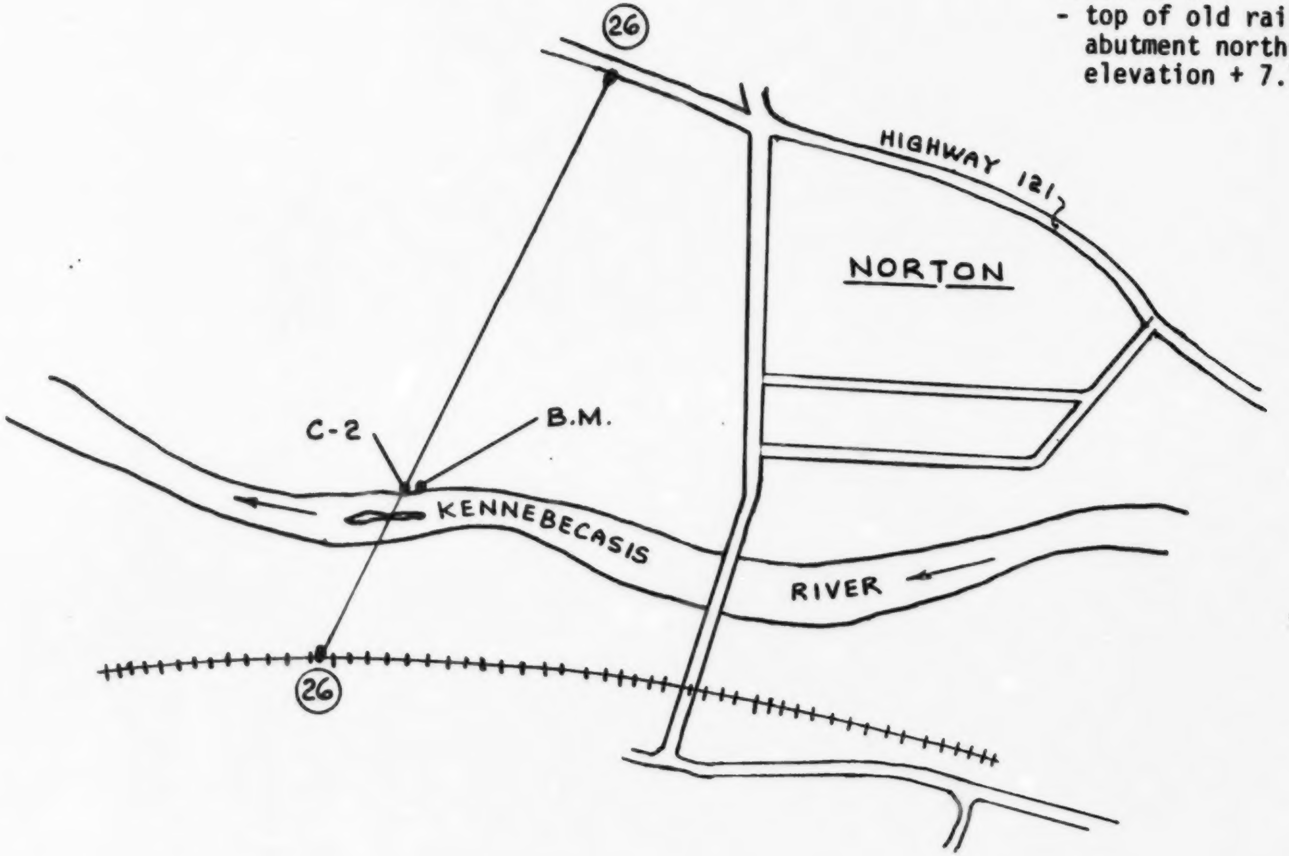

HIGHWAY 121
10
9
8
C-1
KENNEBECASIS RIVER
B.M.
C.N.R.
NORTON
HIGHWAY Nº 2

Figure 7 - 1
Site Plan - Crest Gauge C - 1
N.T.S.

Crest Gauge C - 2
- mounted to old railway bridge abutment

Bench Mark
- top of old railway bridge abutment northside elevation + 7.76 m



㉖
HIGHWAY 121
NORTON
C-2
B.M.
KENNEBECASIS
RIVER
㉖

Figure 7 - 2
Site Plan - Crest Gauge No. 2
N.T.S.

Crest Gauge C - 3
- mounted to a large maple tree 15 m D/S of Bloomfield bridge

Bench Mark
- G.S.C. monument 77B030 elevation + 8.473 m

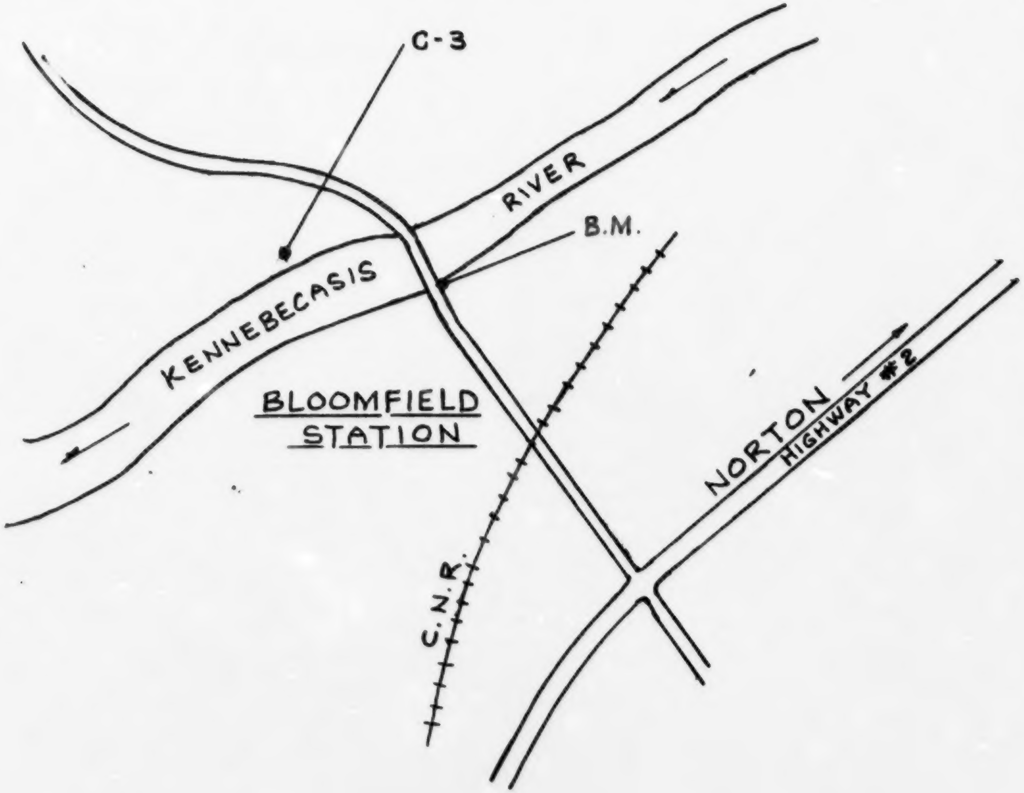

C-3
KENNEBECASIS
RIVER
B.M.
BLOOMFIELD
STATION
C.N.R.
NORTON
HIGHWAY #2

Figure 7 - 3
Site Plan - Crest Gauge C - 3
N.T.S.

Crest Gauge C-4
- mounted on U/S side of bridge east abutment

Bench Mark
- top of bridge abutment
- north east corner
- elevation + 7.09 m

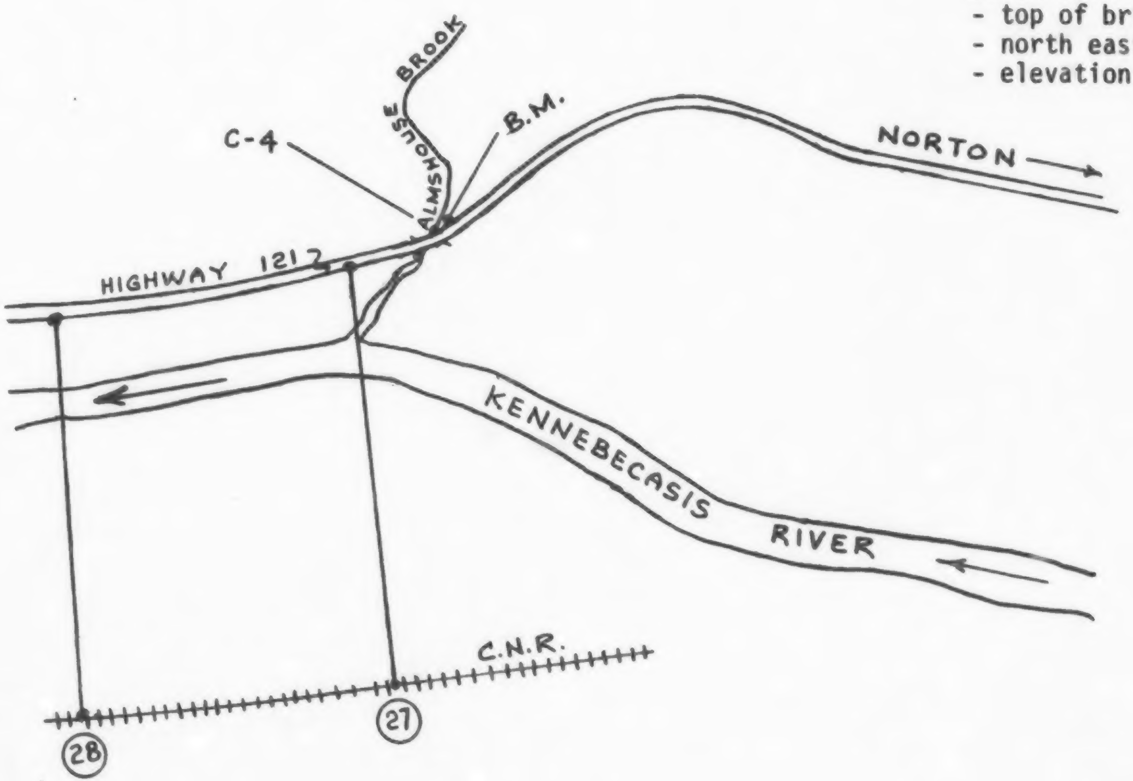

ALMSHOUSE BROOK
C-4
B.M.
NORTON
HIGHWAY 121
KENNEBECASIS RIVER
C.N.R.
(27)
(28)

Figure 7 - 4

Site Plan - Crest Gauge C -4

N.T.S.

Crest Gauge C - 5
- mounted on D/S side of
  east brudge abutment

Bench Mark
- G.S.C. monument 77B027
  elevation + 11.545

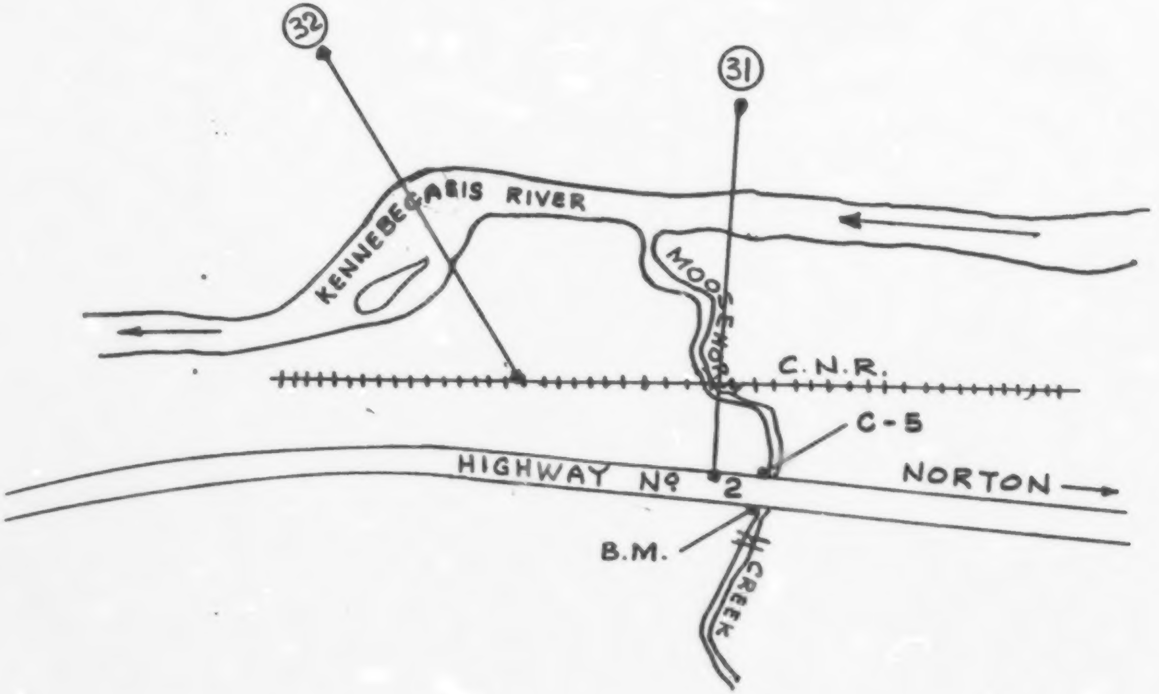

32
31
KENNEBECASIS RIVER
MOOSEHORN
CREEK
C.N.R.
C-5
HIGHWAY Nº 2
NORTON
B.M.

Figure 7 - 5

Site Plan - Crest Gauge C - 5

N.T.S.

flows were all confined within the river bank i.e. no overbank flooding. Of these high river flows, the event of November 14, 1982 was monitored for calibration of the HEC II model.

The high flow flooding event of February 12, 1981, which is well documented, was used as a second calibration flow.

7.2 Calibrating Flows

7.2.1 November 14, 1982

During the period November 13-14, 1982 approximately 30-40 mm of rain fell on the Kennebecasis River watershed producing a recorded peak flow at the Apohaqui gauging station of 92.8 m^3/sec. The hydrograph of this event is shown in Figure 9. The high flow produced water levels in the Kennebecasis River in the study area at or near the top of the river bank.

On November 14, 1982 between 1030 and 1215, the Geocon Inc. field crew marked, with stakes, the water surface elevation at various points throughout the study reach. Subsequently the marks on these stakes were levelled to geodetic datum by the Fenco field crew.

Water surface elevations for this event were taken at either bridge or easily accessible locations throughout the study area. A summary of location, time and elevations of water surface measurements during this event are given in Table 7-2. The water surface profile of this event is shown on Figures 7-8, and 7-9.

The water levels recorded on the Kennebecasis River are considered peak levels. However at the time of monitoring, levels had dropped approximately 0.5 m and 0.3 m from peak levels on the Almshouse Brook and Moosehorn Creek respectively.

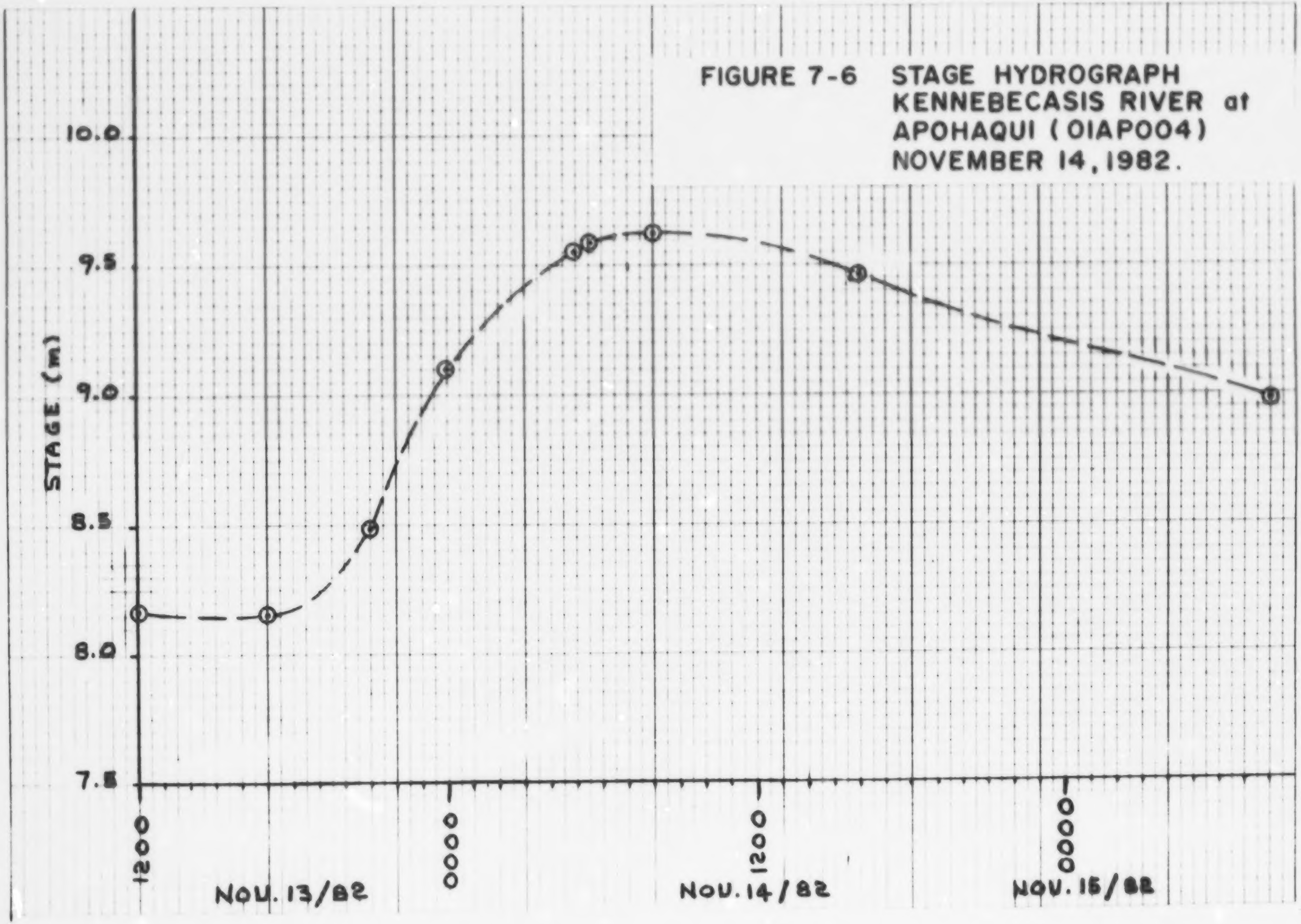


FIGURE 7-6 STAGE HYDROGRAPH
KENNEBECASIS RIVER at
APOHAQUI (01AP004)
NOVEMBER 14, 1982.

TABLE 7-2

WATER SURFACE ELEVATIONS

NOVEMBER 14, 1982

RIVER COURSE	LOCATION	TIME OF MEASUREMENT	RIVER ELEVATION (m)	MAX. WATER ELEVATION (m)
Kennebecasis River	Apohaqui Bridge (B-1) Section 1	1020	9.2	9.2
Kennebecasis River	Section 13	1215	7.5	7.5
Kennebecasis River	Old Railway Bridge Norton Section 26	1030	5.2	5.2
Kennebecasis River	Bloomfield Bridge	1110	3.0	3.0
Moosehorn Creek	Highway No. 1 Bridge (B-5)	1120	6.2	6.5
Almshouse Brook	Highway No. 121 Bridge (B-4)	1100	5.1	5.6

7.2.2 February 12, 1981

Originally, we had hoped to monitor a flooding event during the spring of 1983 for calibrating data. However, this event did not occur as expected. For calibration purposes it was decided to use the event of February 12, 1981.

On February 12, 1981, a winter storm dropped in excess of 40 mm of rain in the Kennebecasis River watershed. This coupled with a large snowmelt produced a very severe flood in the Norton area. The estimated peak flow at the Apohaqui gauging station was 651 m^3/sec. The hydrograph of this event, which has been reproduced from the "Hydrotechnical Study - Sussex Area Flood Plain" by ADI Ltd., is shown on Figure 7-7.

The water surface elevations of the Kennebecasis River at various locations during this event are given in Table 7-3. These elevations have been interpolated from the high quality aerial photographs of the event supplied to us by Environment New Brunswick. These photographs were taken in the afternoon of February 12, 1981 which, from all acounts, appears to be at or near peak river elevations.

The water surface profile of this event is shown on Figures 7-8 and 7-9.

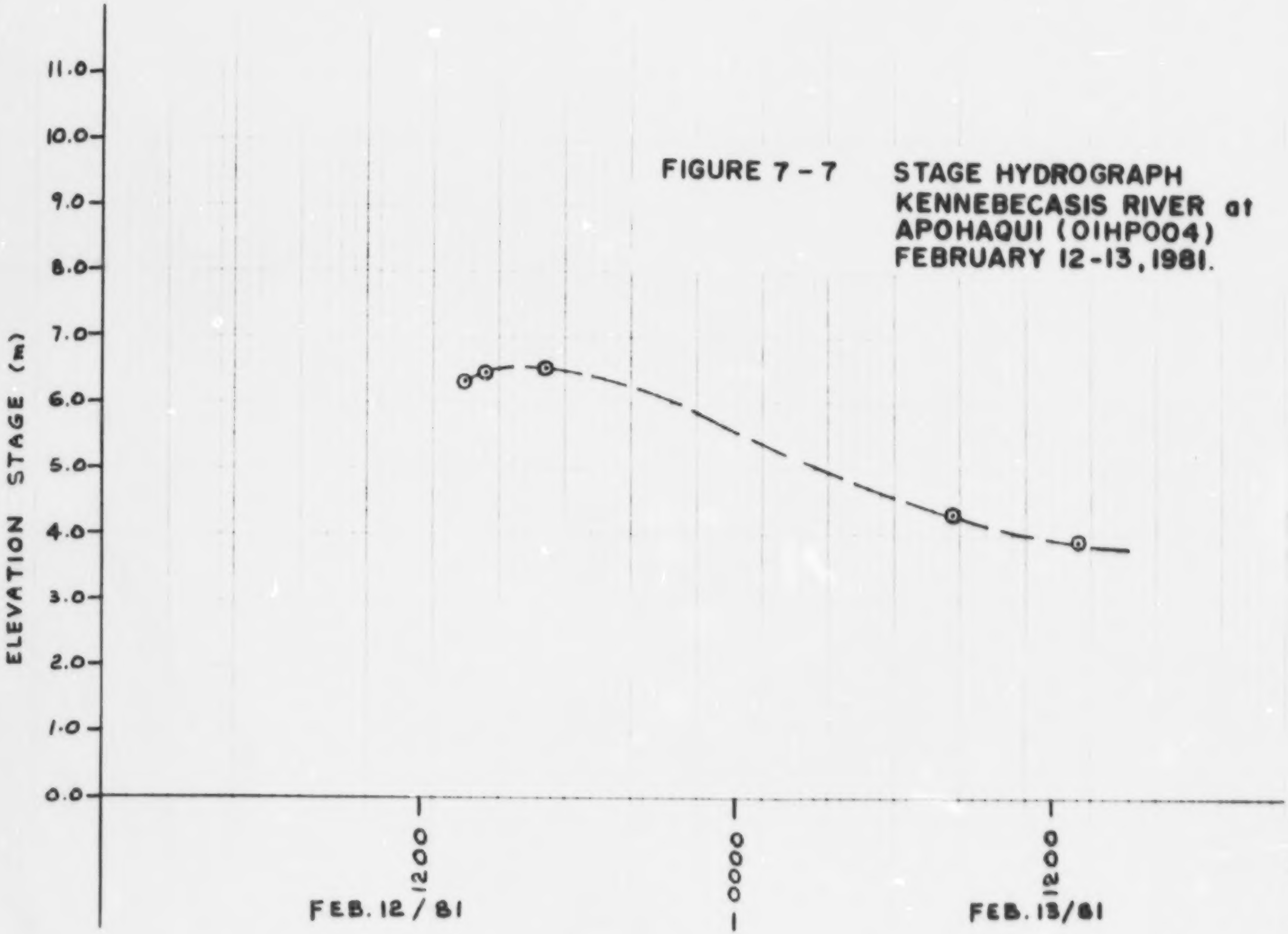


FIGURE 7 - 7 STAGE HYDROGRAPH
KENNEBECASIS RIVER at
APOHAQUI (01HP004)
FEBRUARY 12-13, 1981.

TABLE 7-3

WATER SURFACE ELEVATIONS

FEBRUARY 12, 1981

LOCATION	MAX. WATER ELEVATION (m)
Apohaqui Bridge (B-1) Section 1	13.8
Norton Bridge (B-3) Section 25.3	8.65
Bloomfield Bridge Section 36.3	6.7
Hampton Bridge Section 40.3	4.4

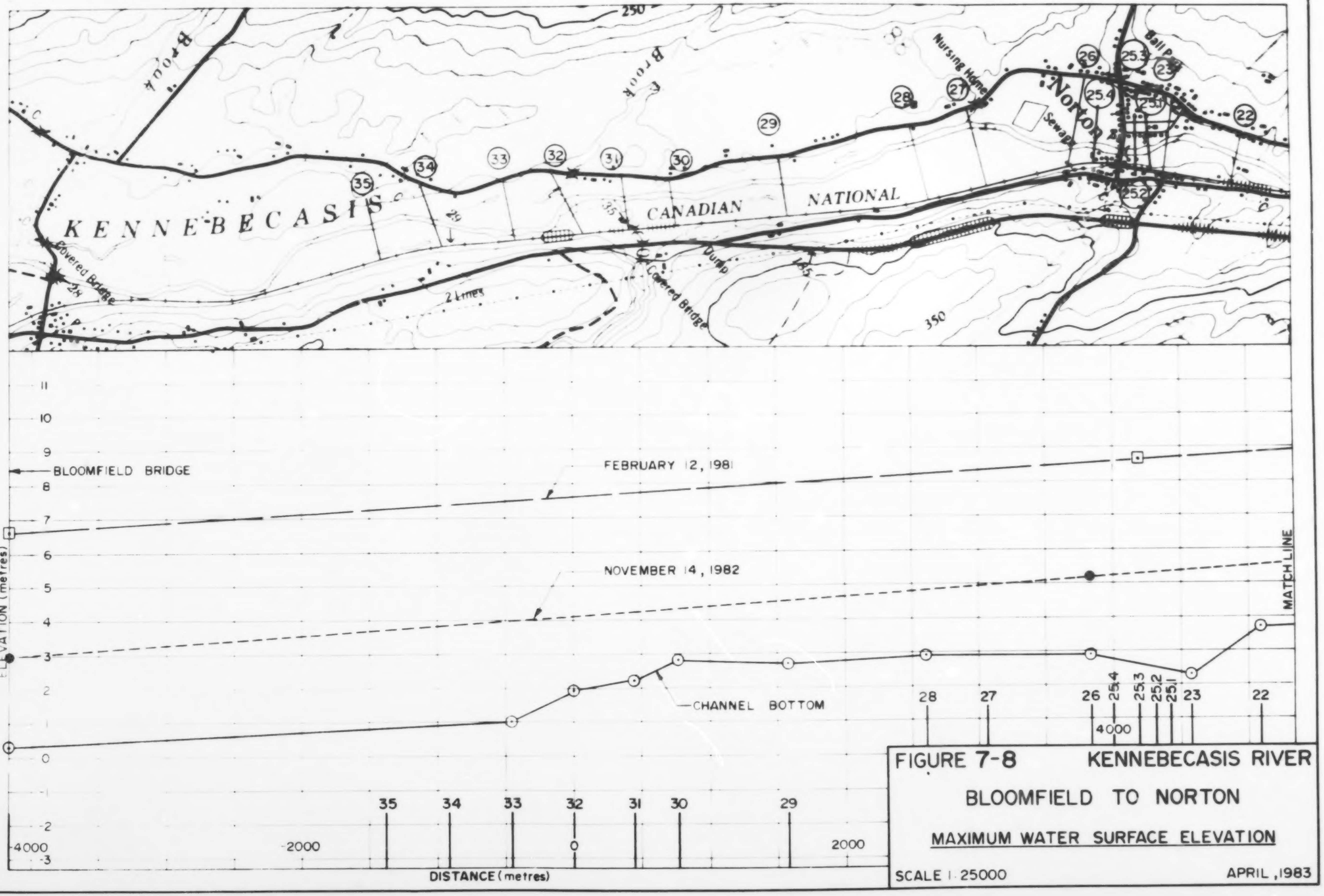


FIGURE 7-8 KENNEBECASIS RIVER

BLOOMFIELD TO NORTON

MAXIMUM WATER SURFACE ELEVATION

SCALE 1:25000 APRIL, 1983

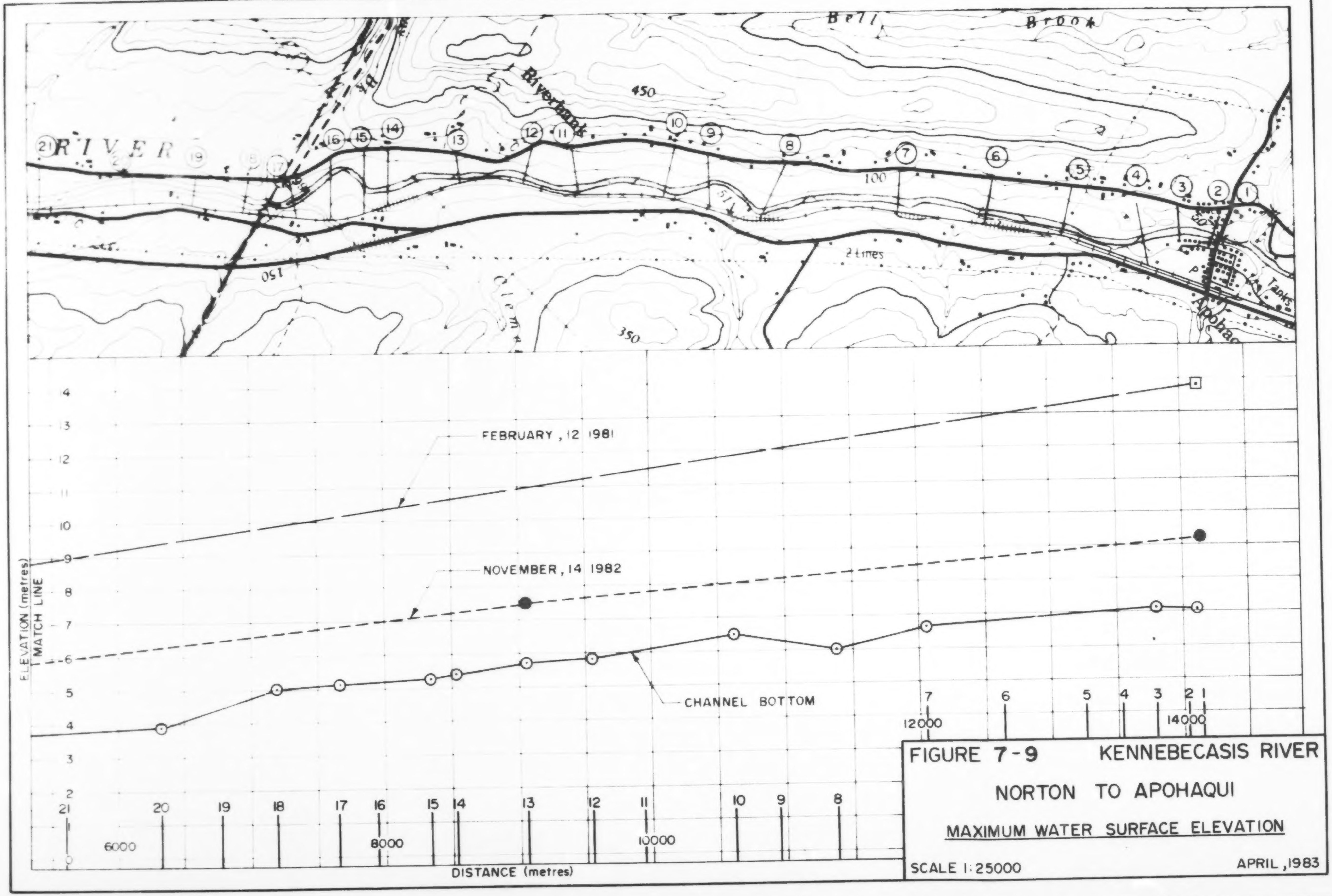


FIGURE 7-9 KENNEBECASIS RIVER

NORTON TO APOHAQUI

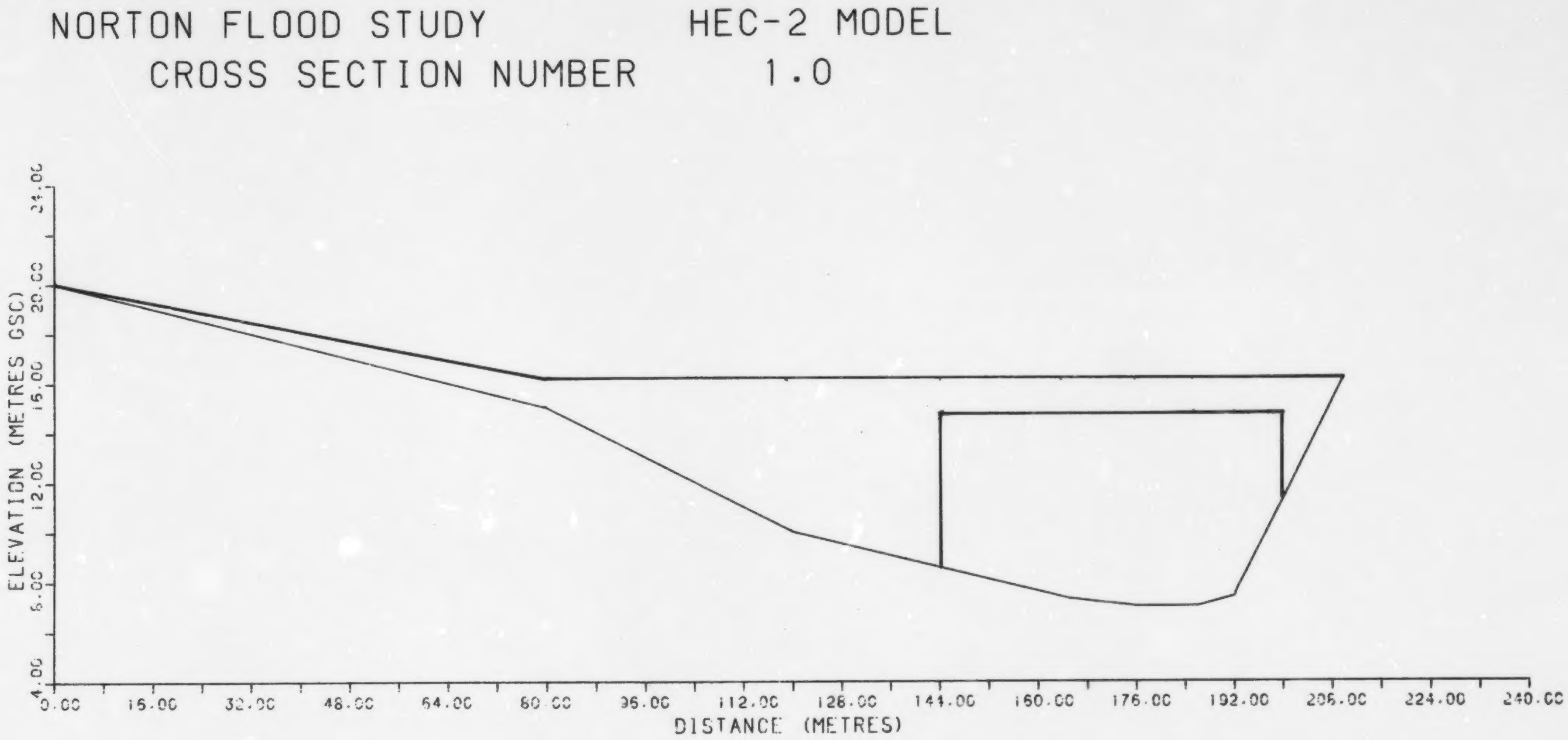
MAXIMUM WATER SURFACE ELEVATION

SCALE 1:25000

APRIL, 1983

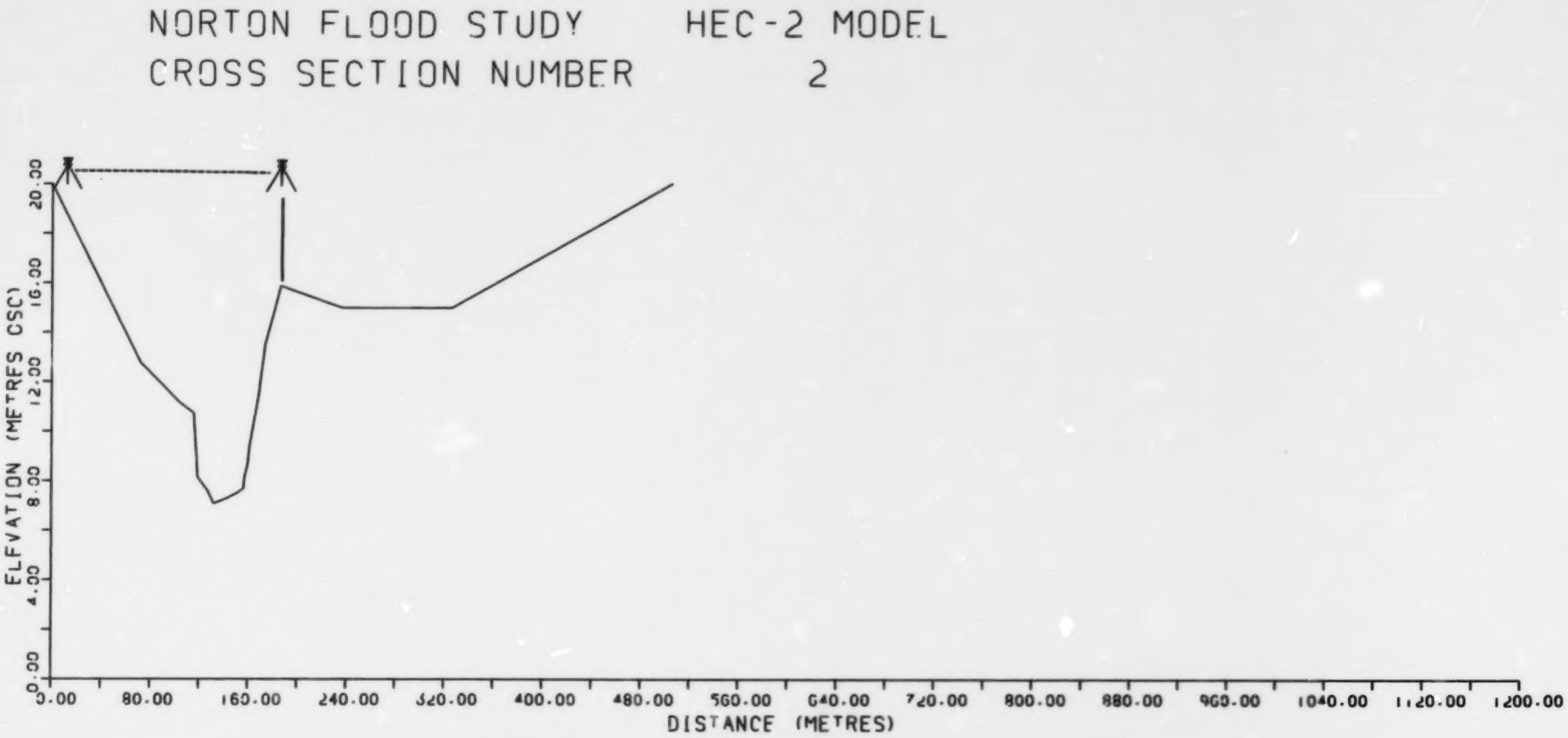
APPENDIX A

PLOTTED CROSS SECTIONS - STUDY AREA



NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 1.0

FIGURE A-1



NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 2

FIGURE A-2

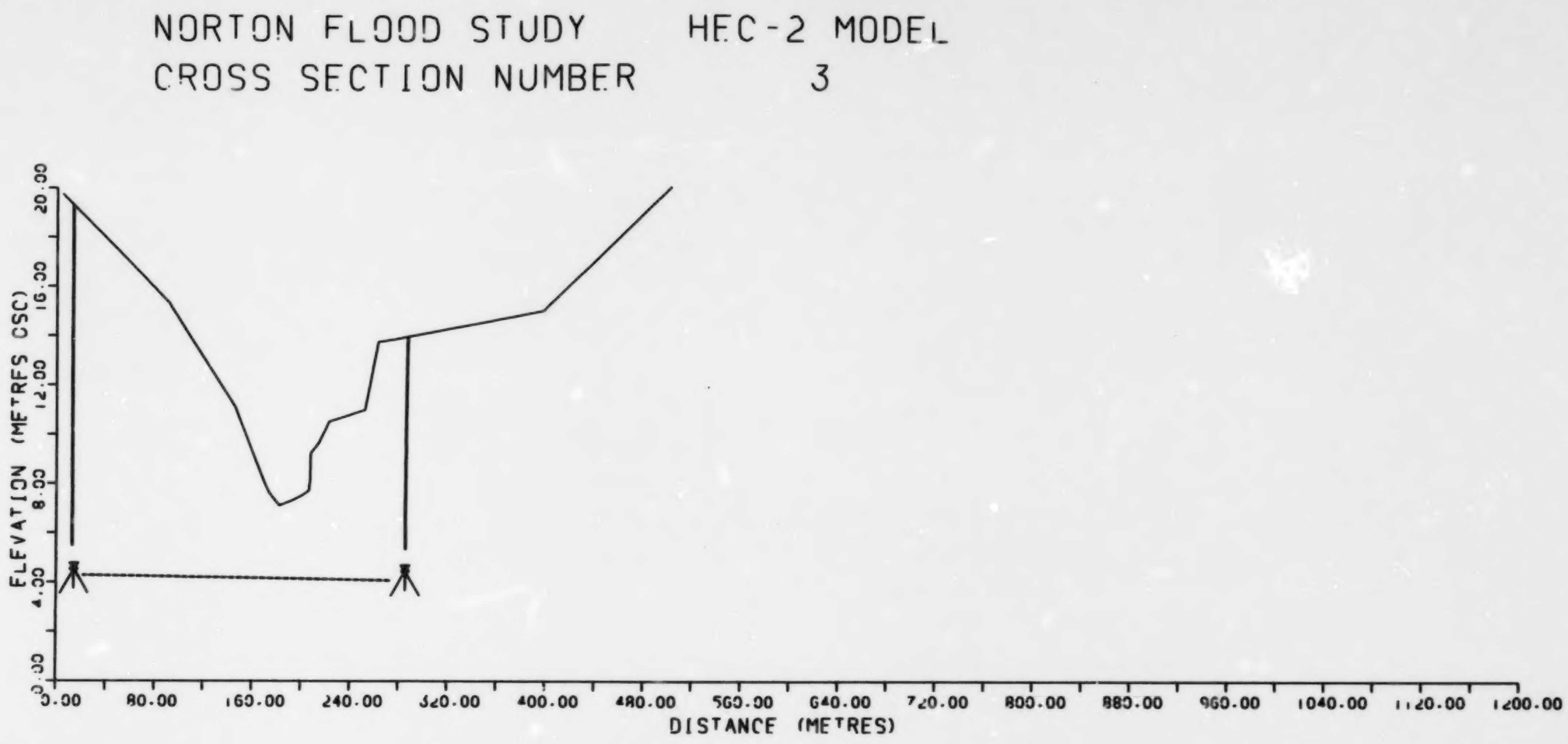

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 3
ELEVATION (METRES GSC)
0.00
4.00
8.00
12.00
16.00
20.00
DISTANCE (METRES)
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00

FIGURE A-3

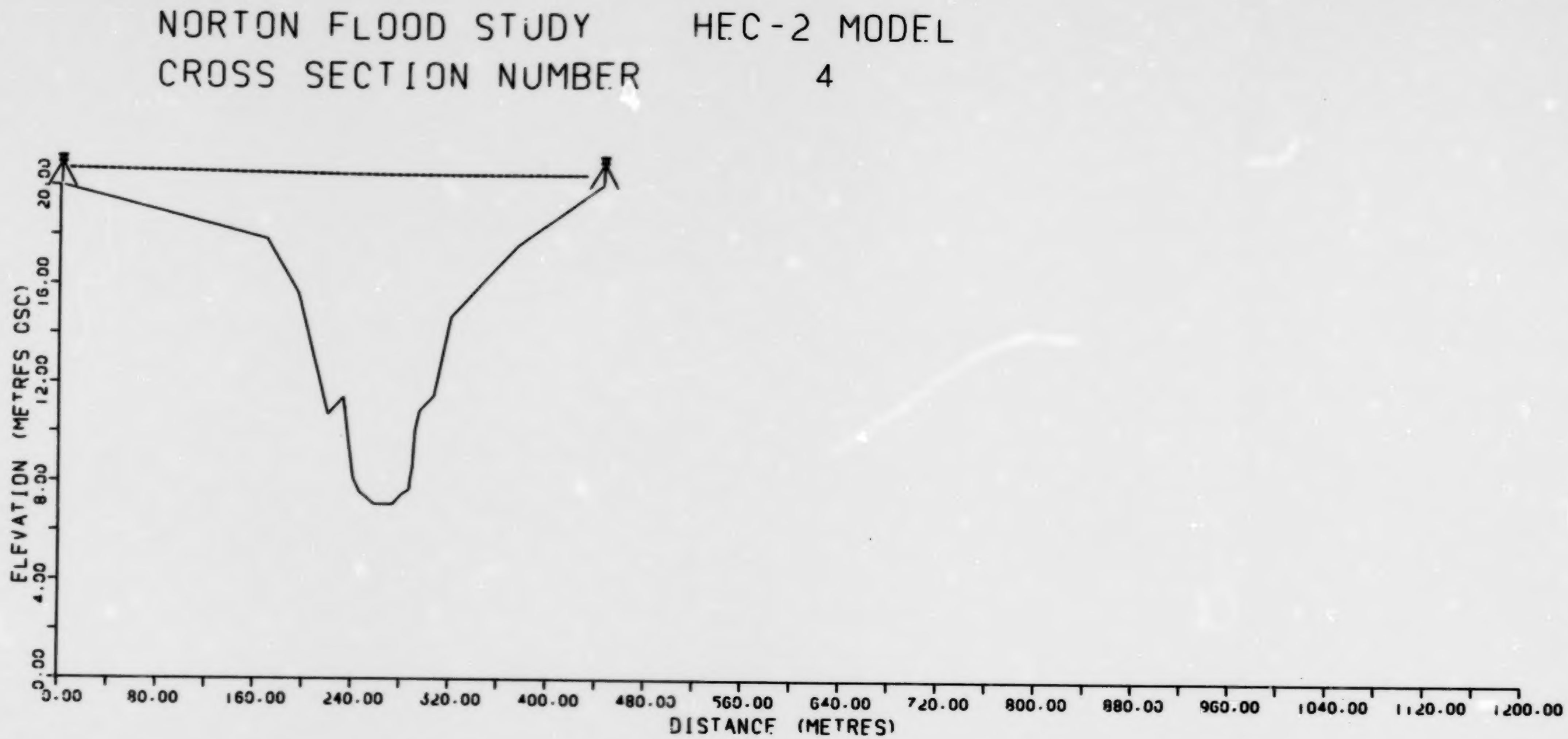
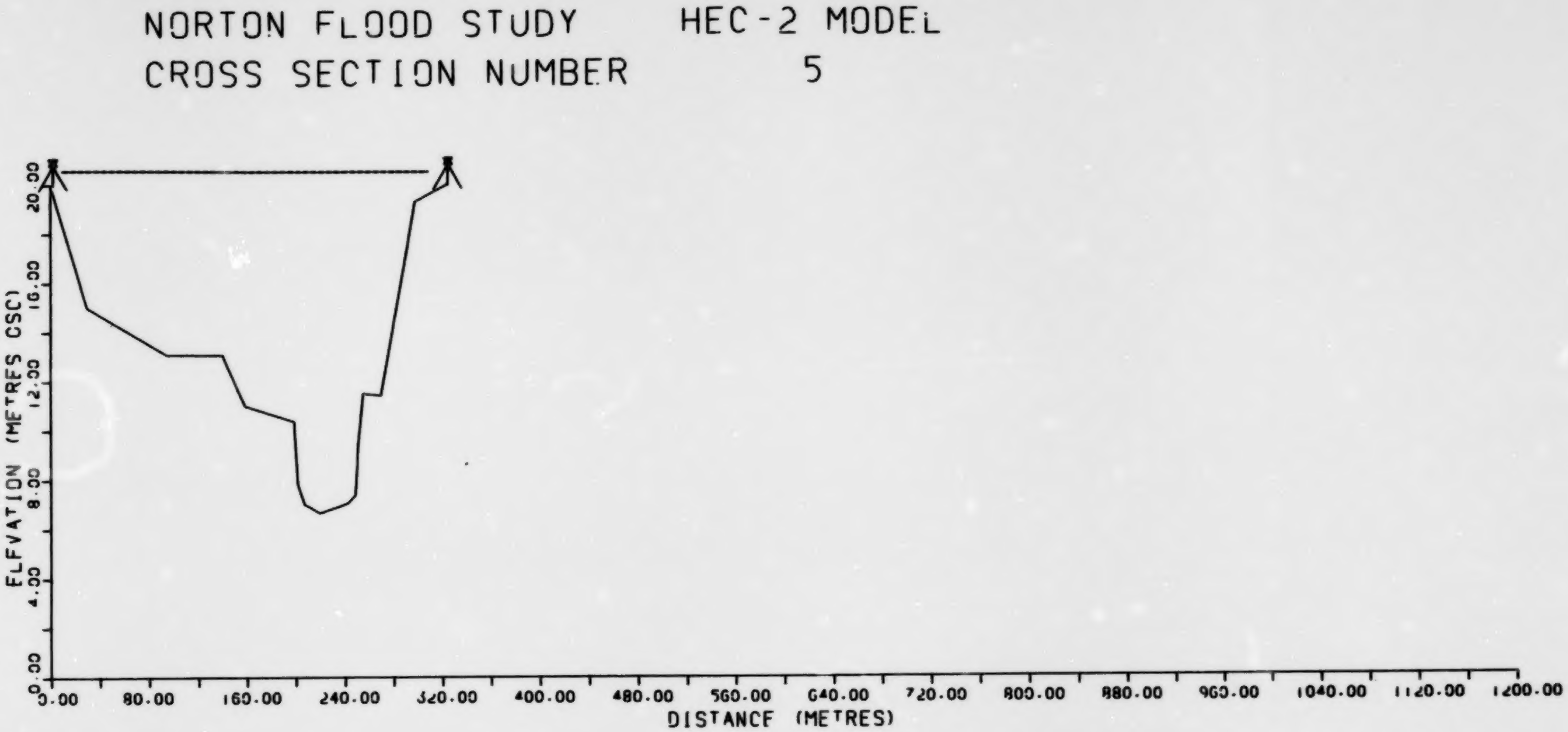
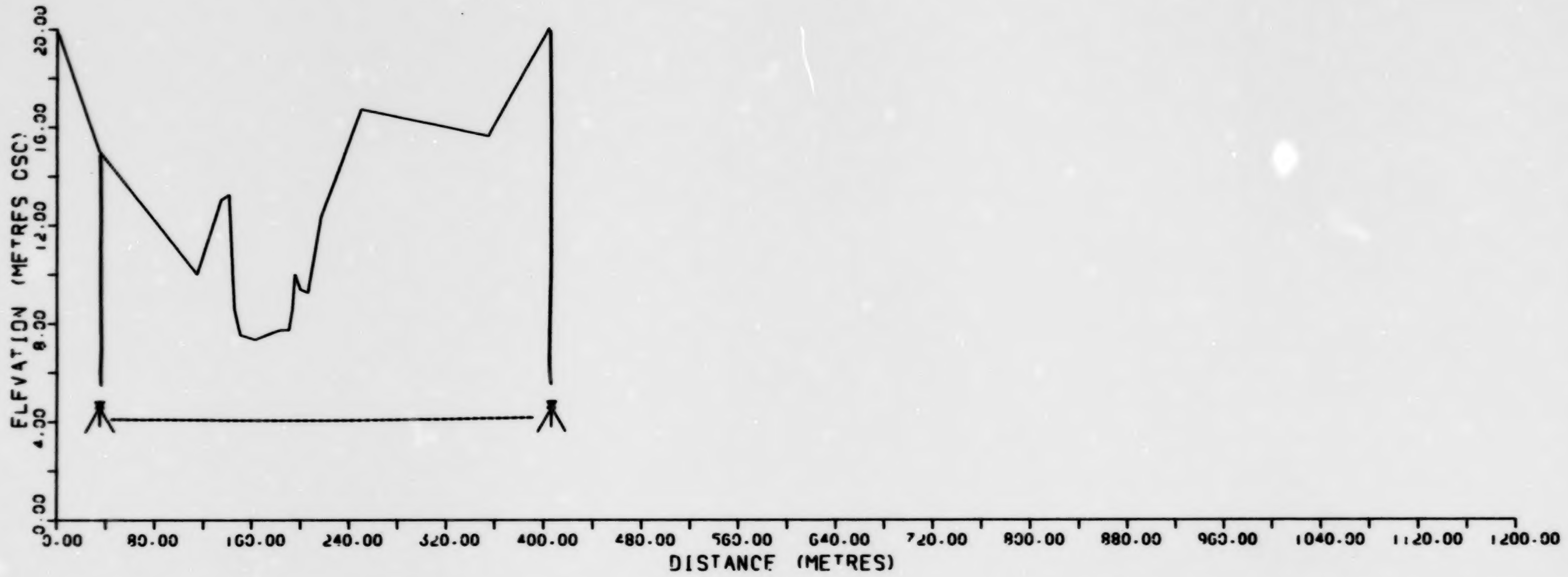

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 4
ELEVATION (METRES GSC)
0.00
4.00
8.00
12.00
16.00
20.00
DISTANCE (METRES)
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00

FIGURE A-4



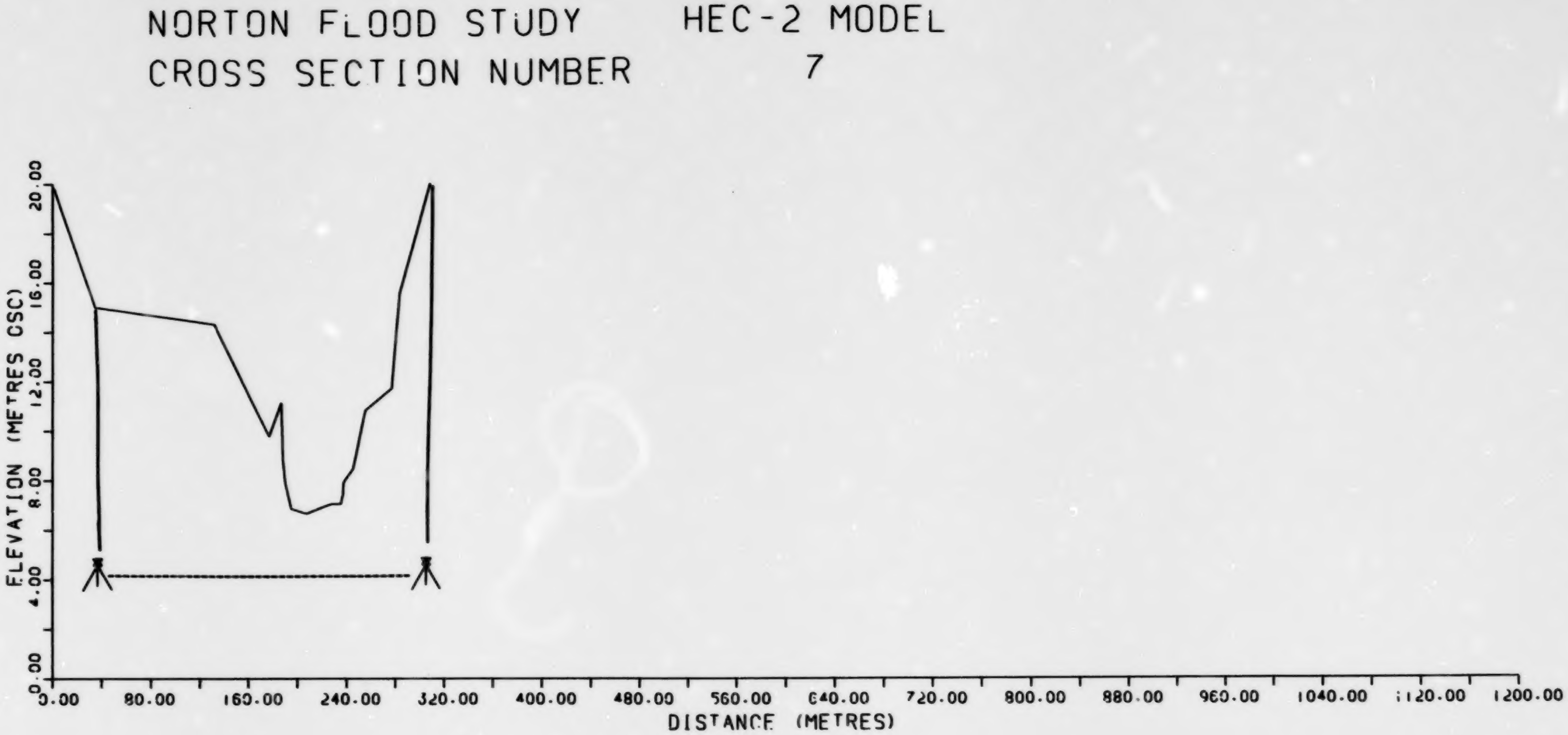
NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 5

FIGURE A-5



NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 6

FIGURE A-6



NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 7

FIGURE A-7

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 8

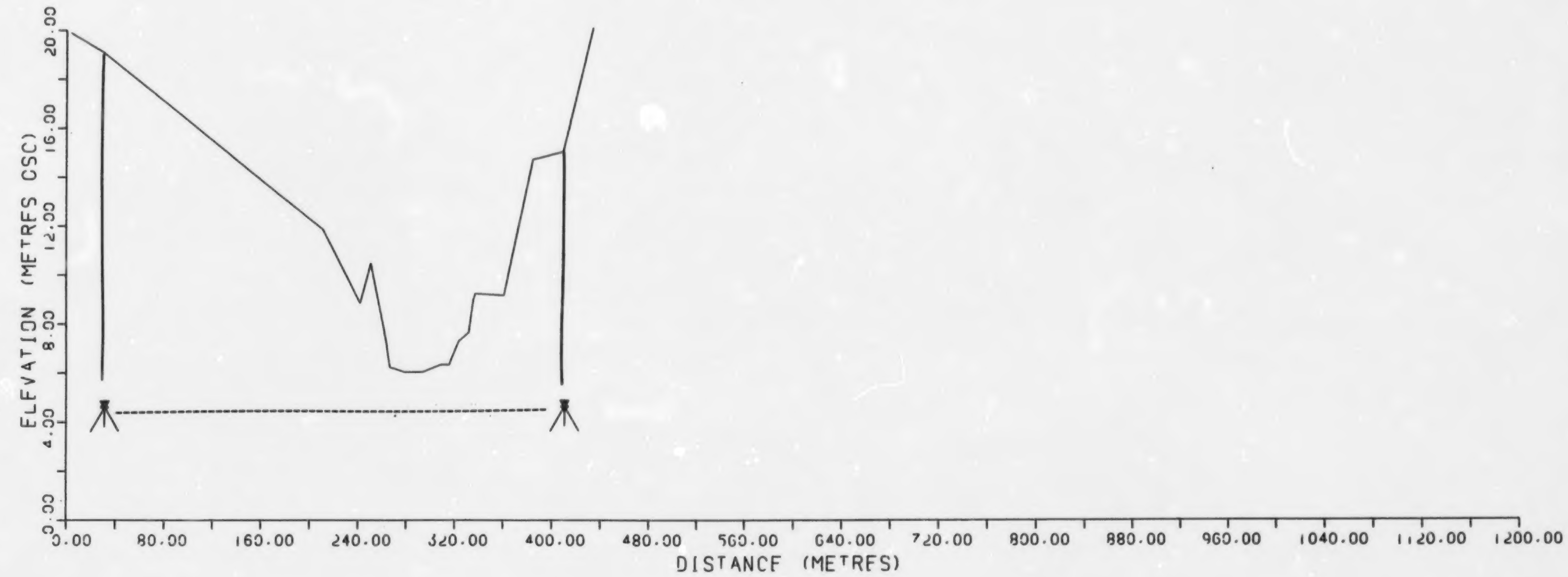


FIGURE A-8

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 9

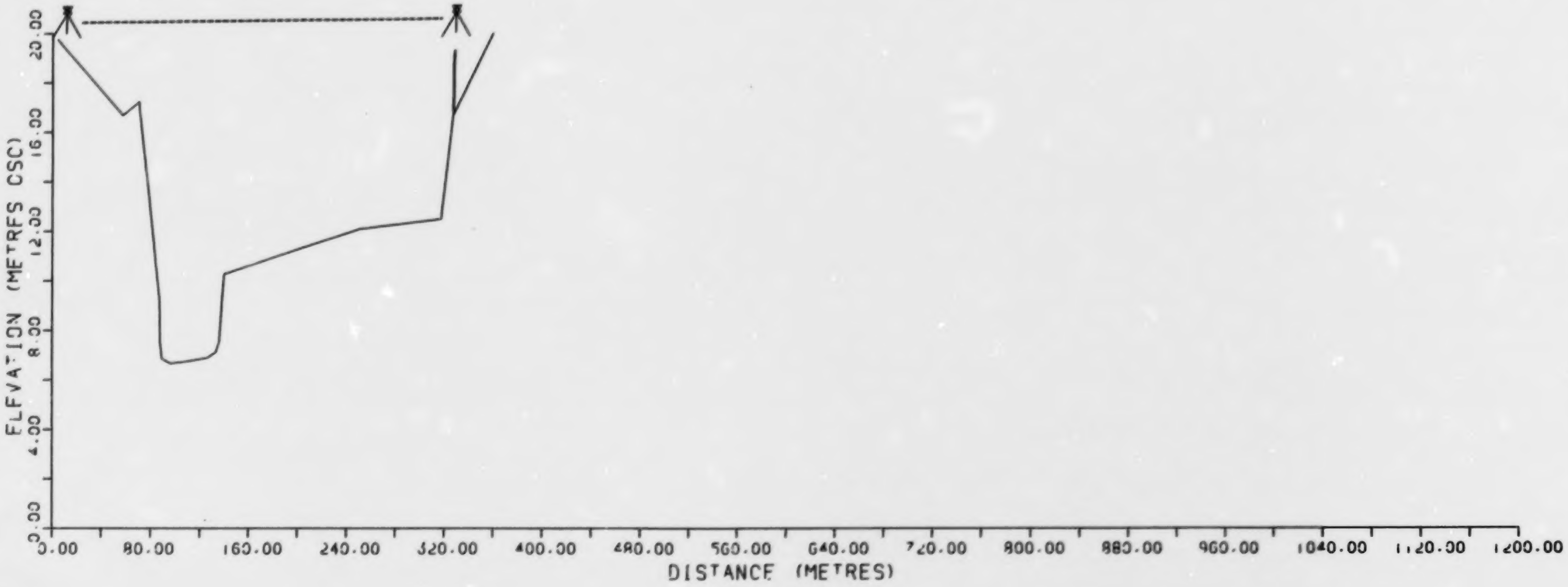


FIGURE A-9

NORTON FLOOD STUDY HEC-2 MODEL

CROSS SECTION NUMBER 10

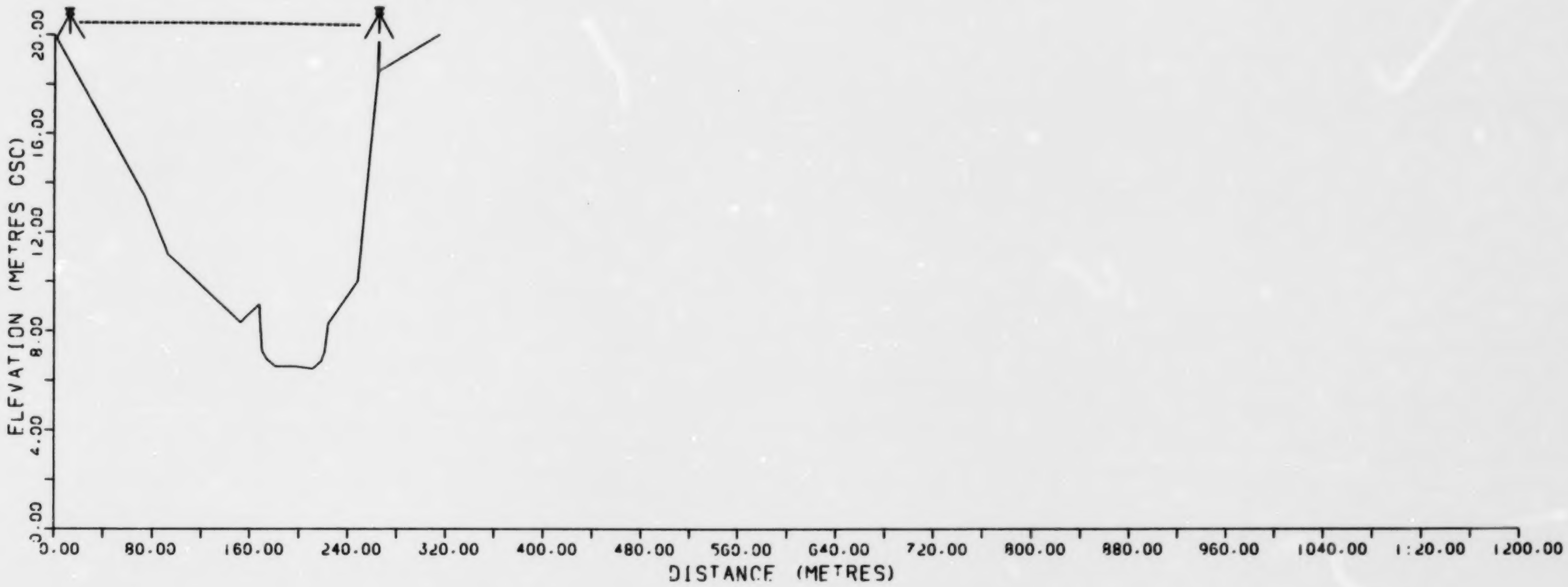


FIGURE A-10

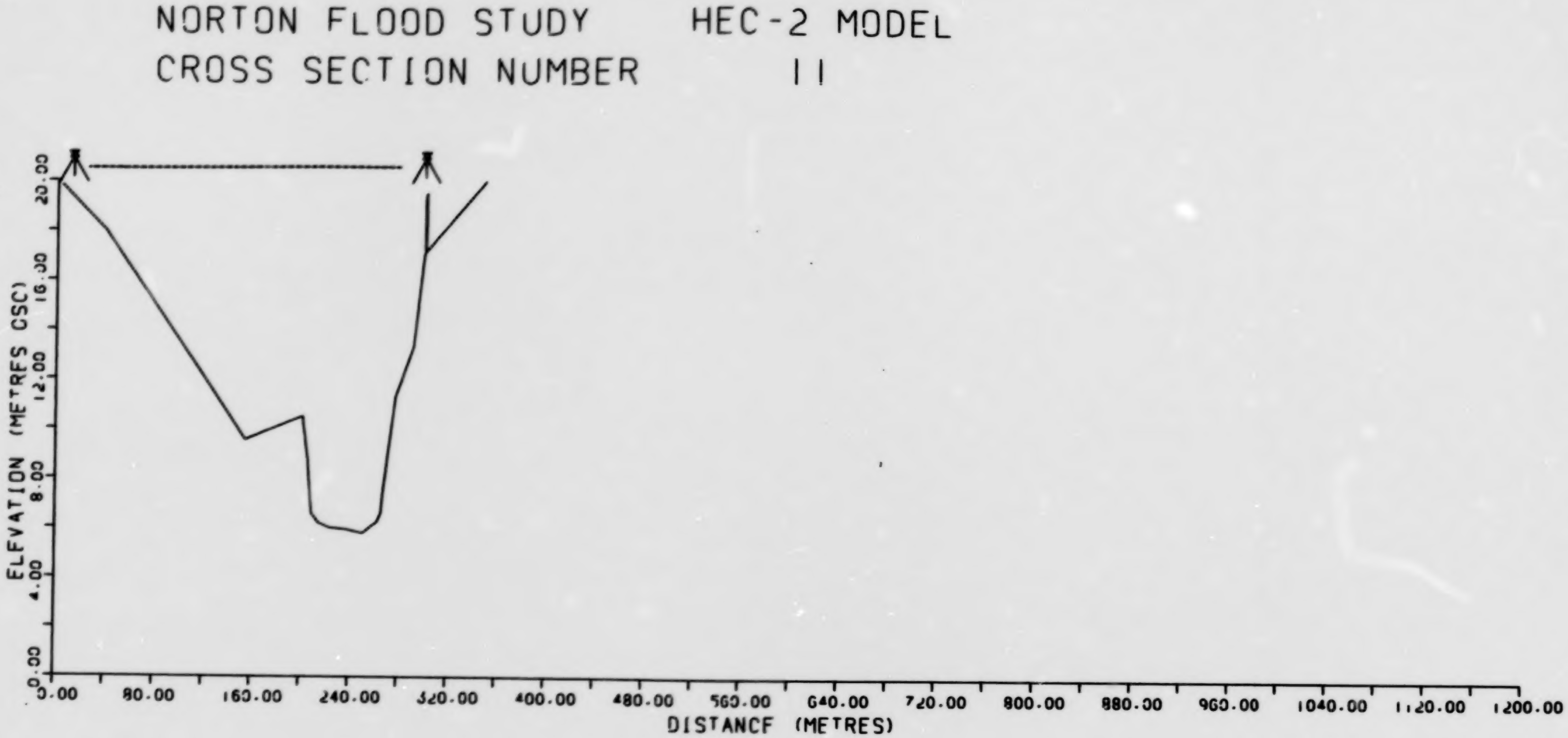

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 11
ELEVATION (METRES CSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE A-11

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 12

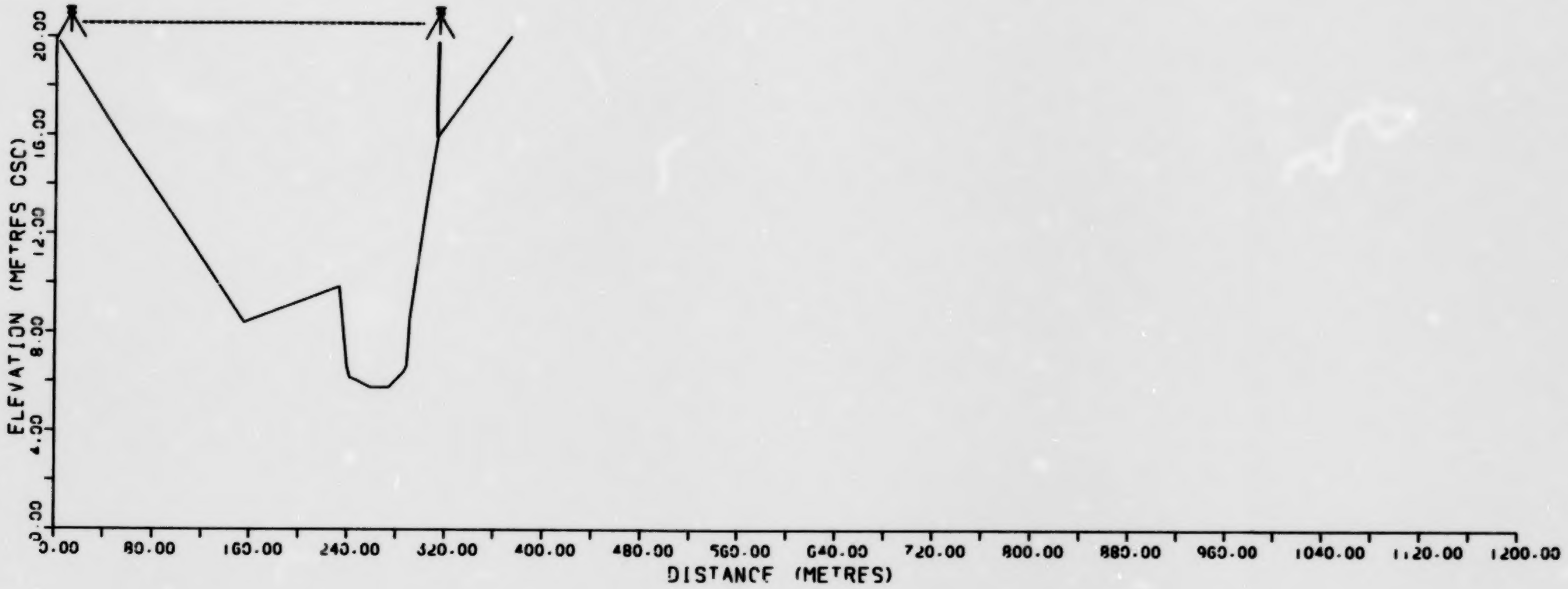


FIGURE A-12

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 13

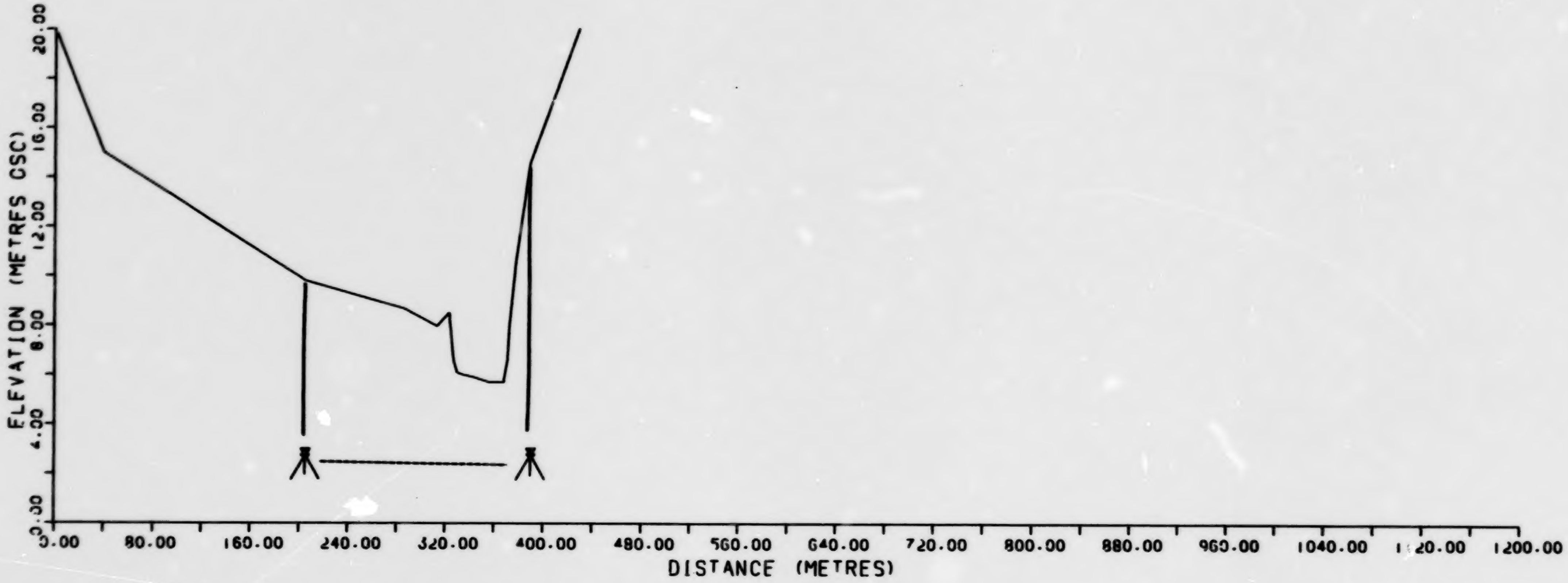


FIGURE A-13

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 14

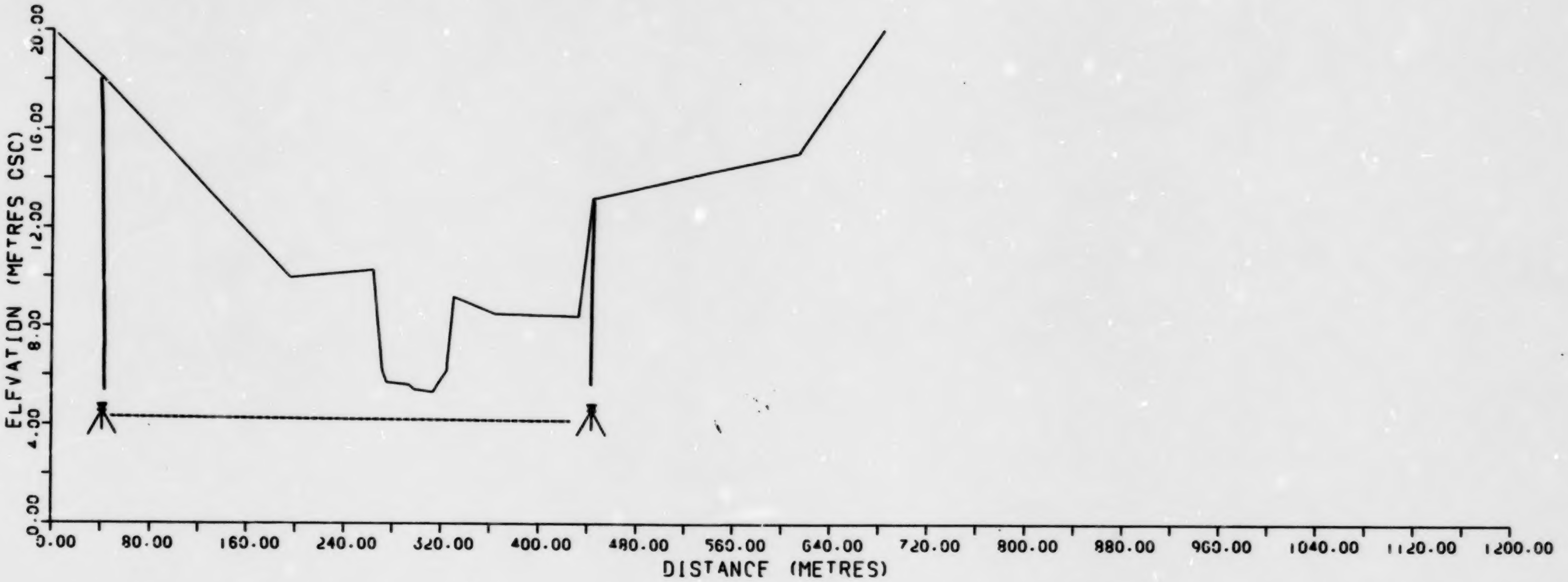


FIGURE A-14

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 15

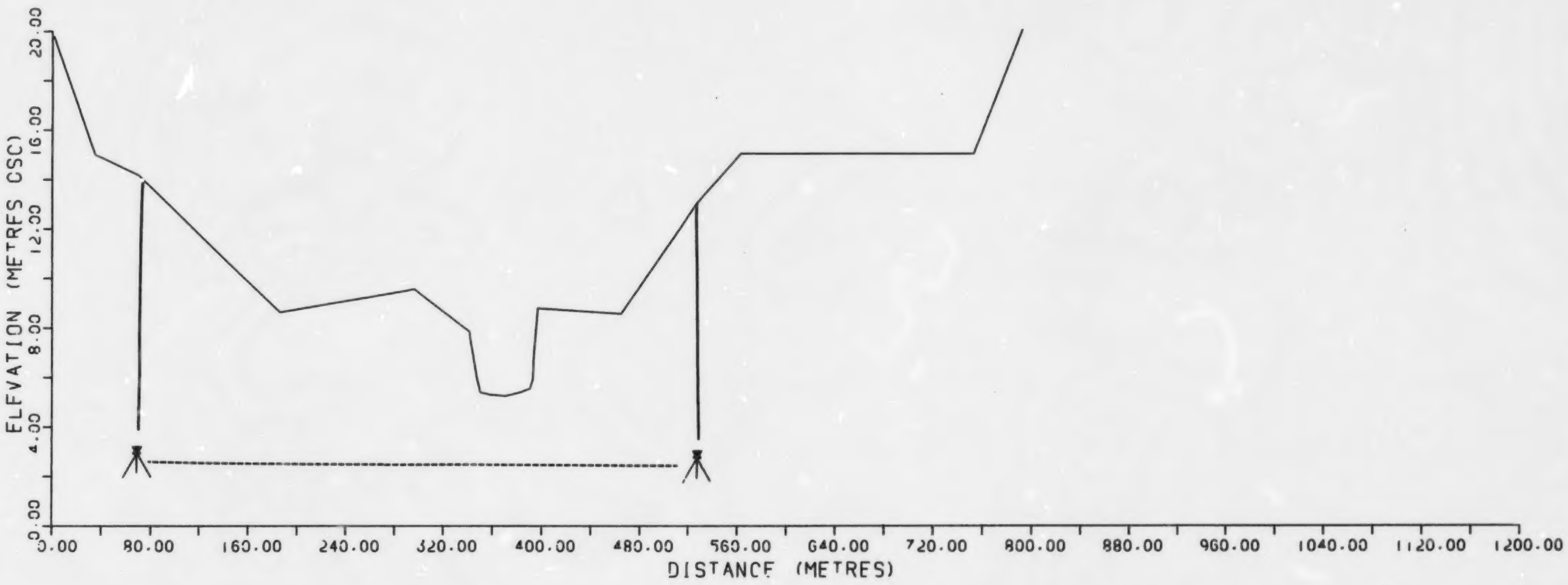


FIGURE A-15

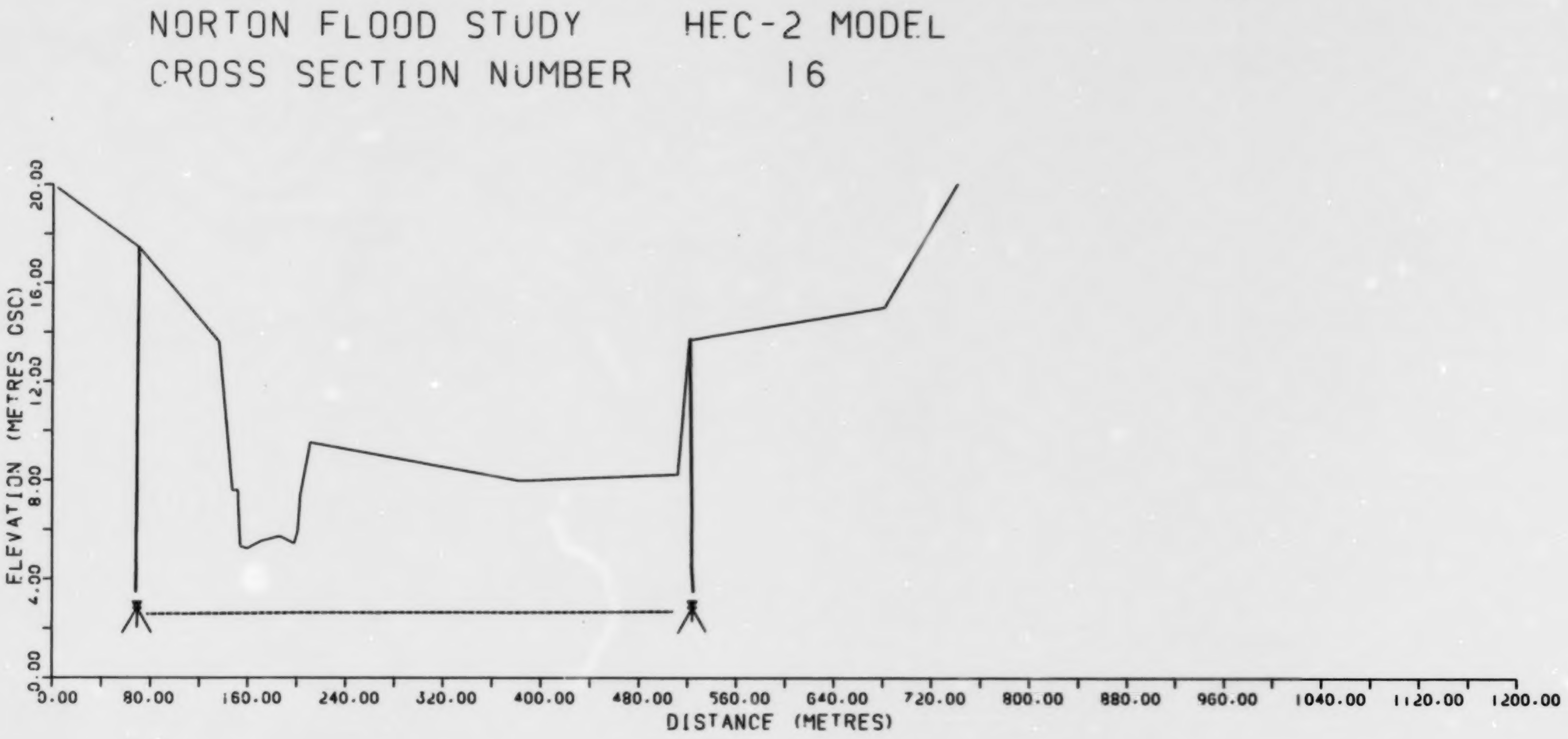

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 16
ELEVATION (METRES GSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE A-16

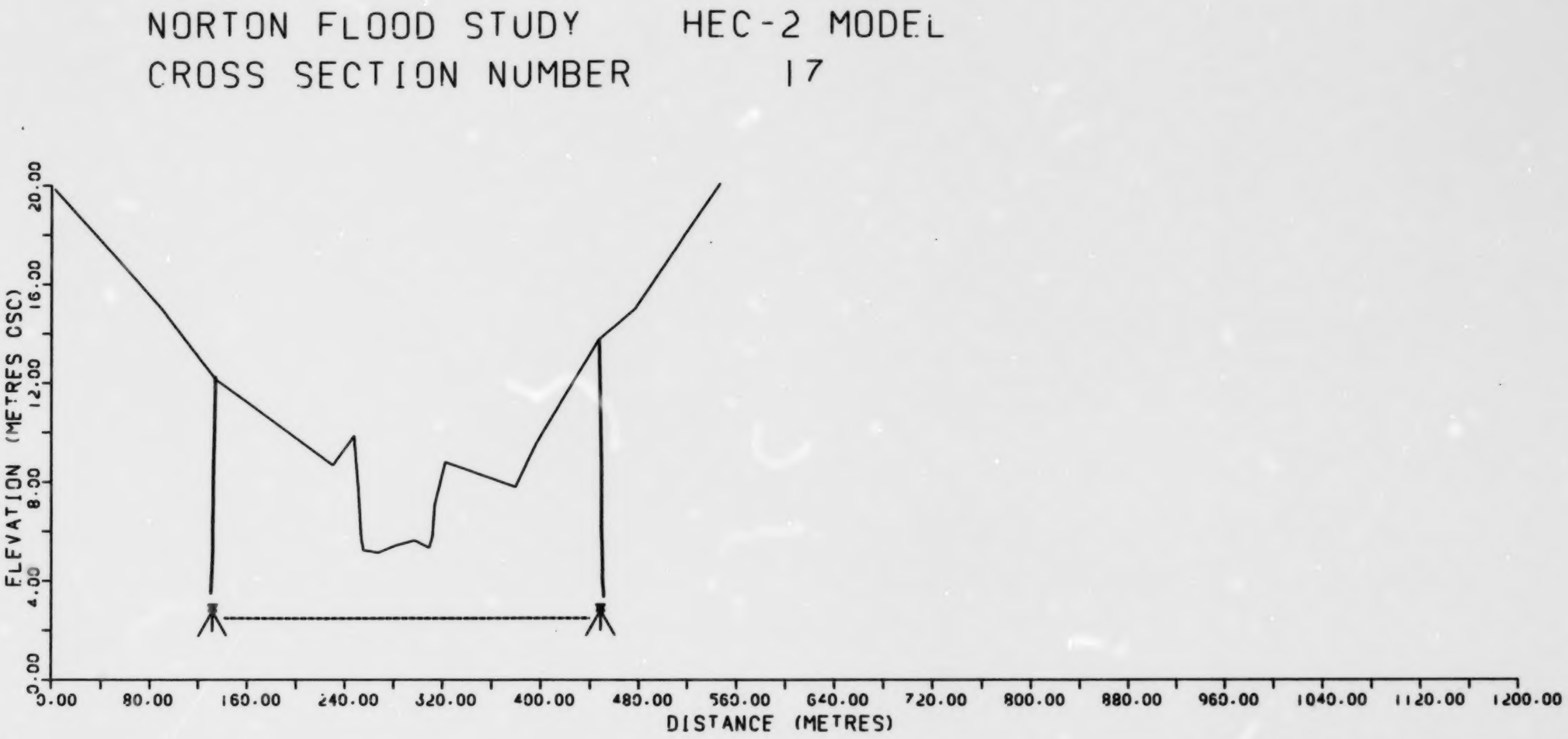

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 17
ELEVATION (METRES CSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE A-17

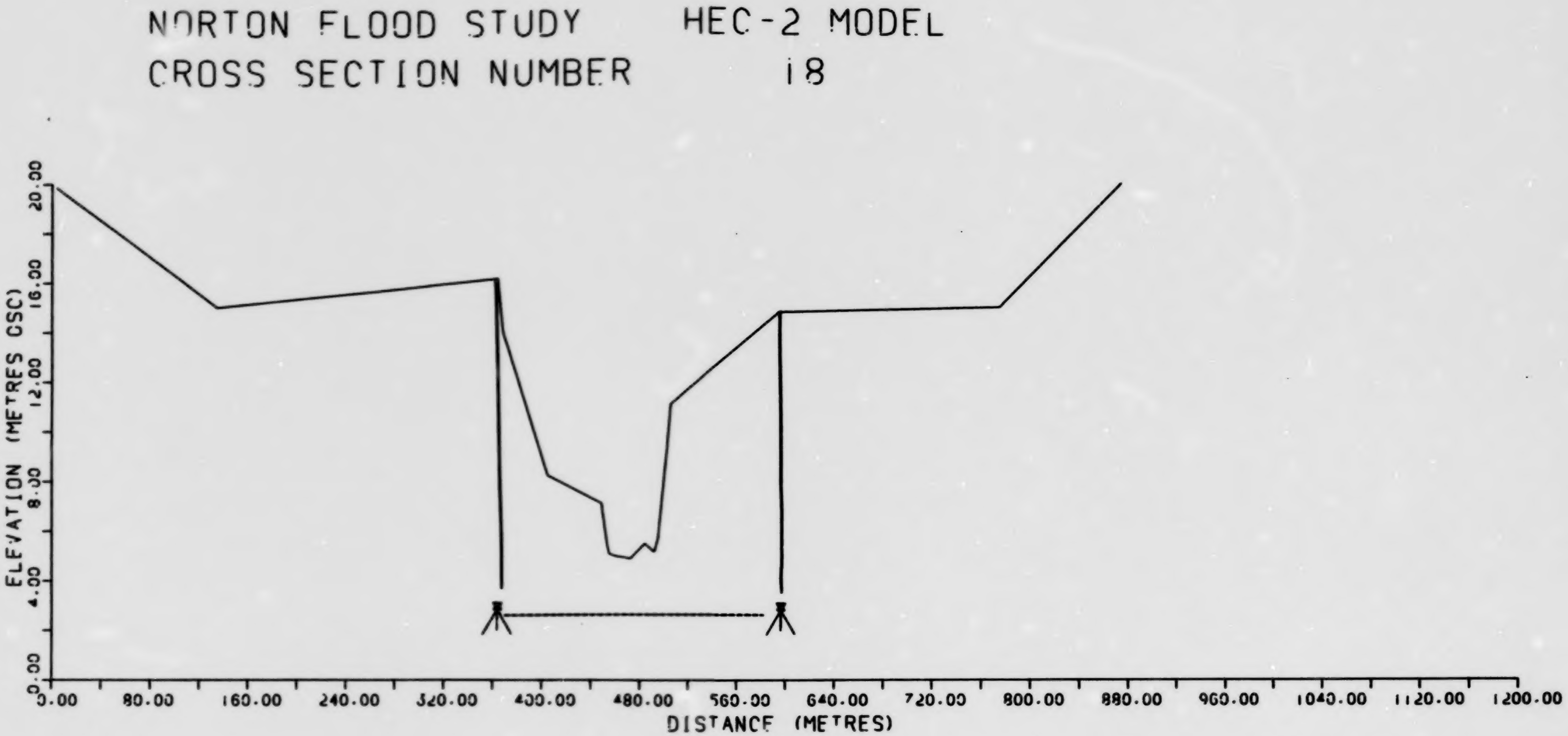

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 18
ELEVATION (METRES GSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE A-18

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 19

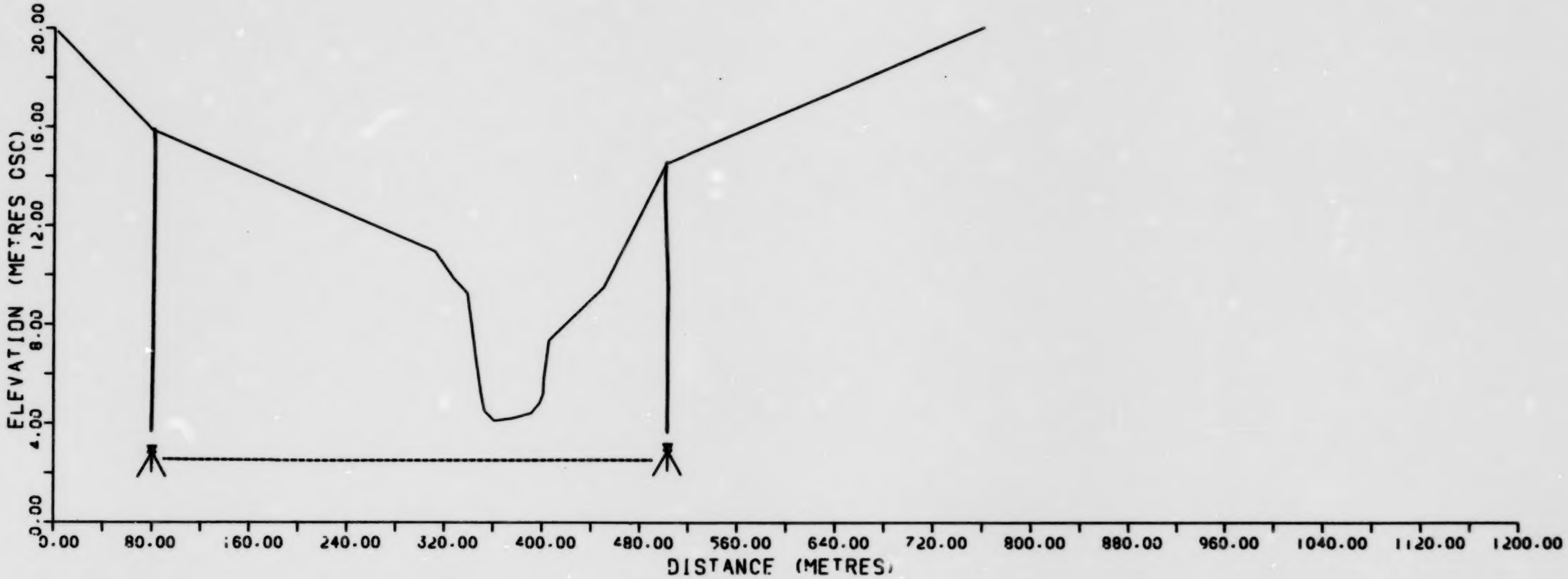


FIGURE A-19

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 20

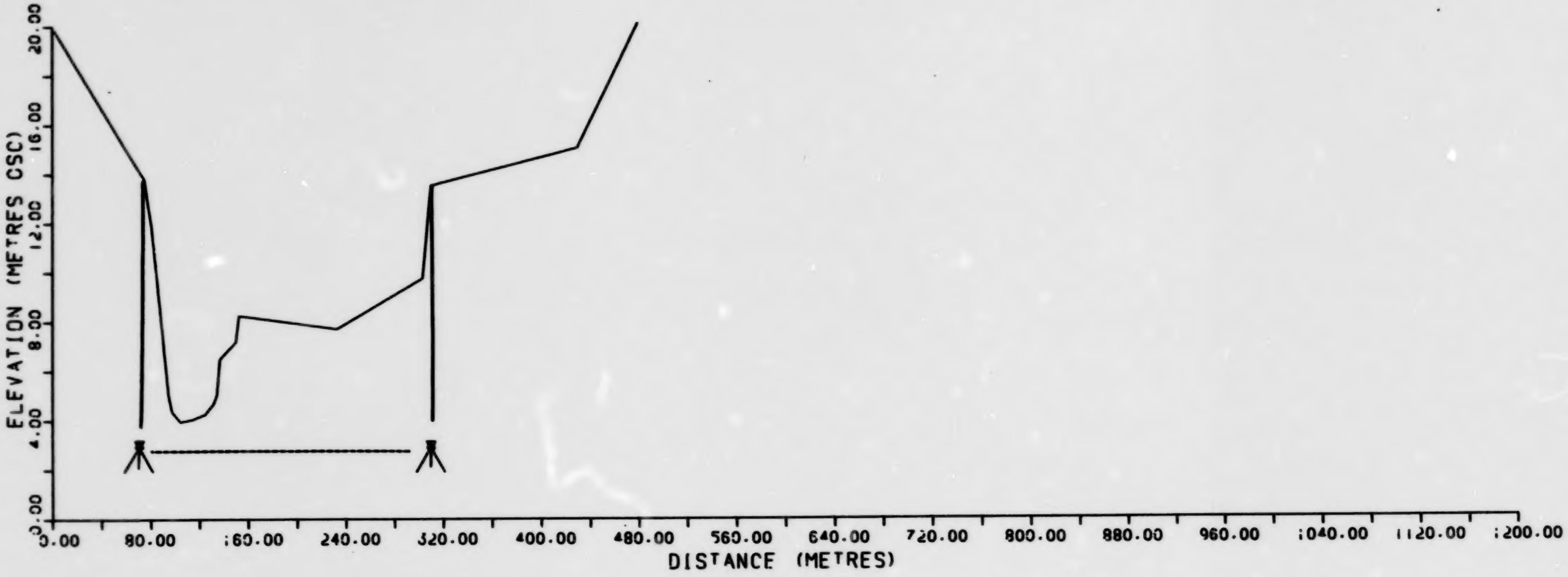
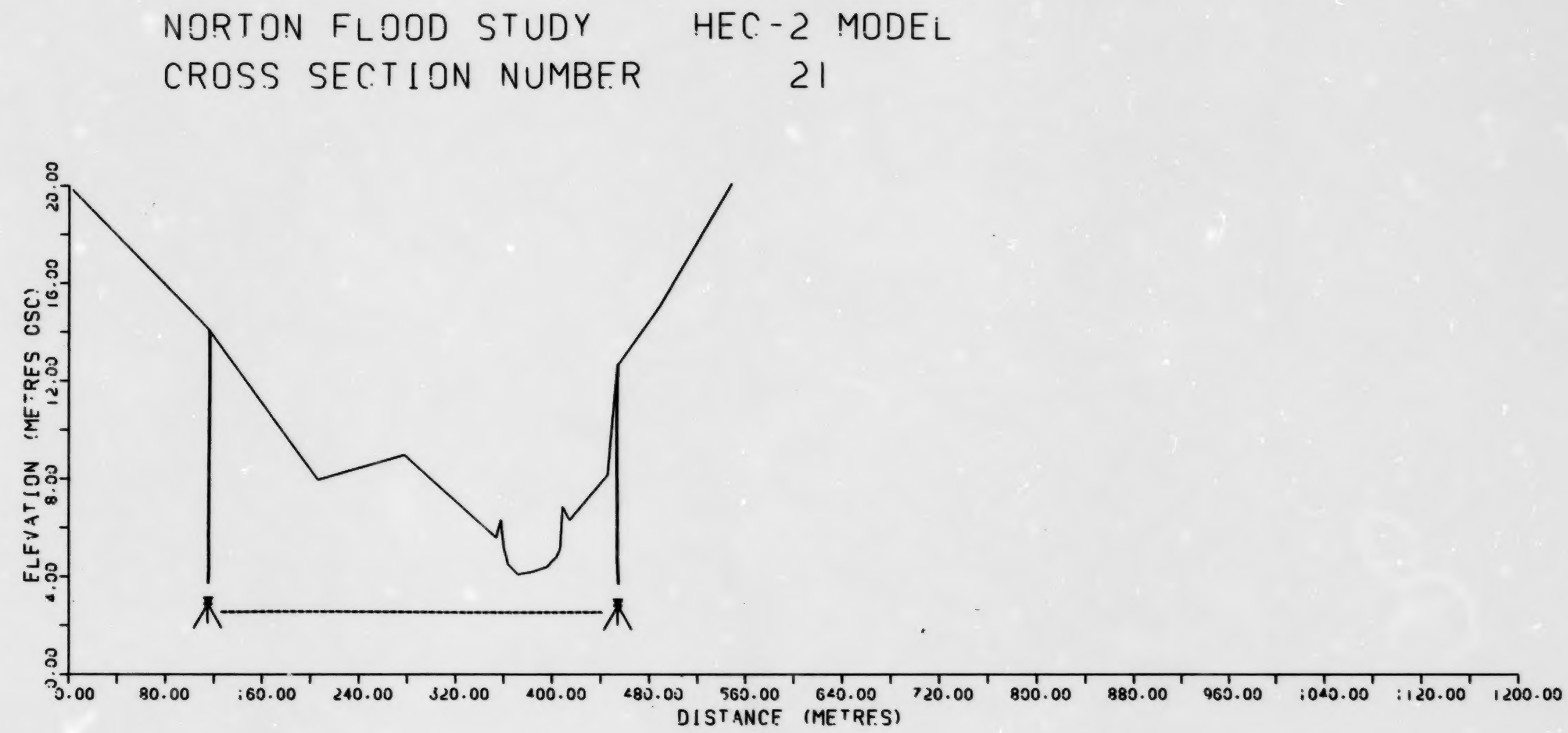


FIGURE A-20



NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 21
ELEVATION (METRES CSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE A-21

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 22

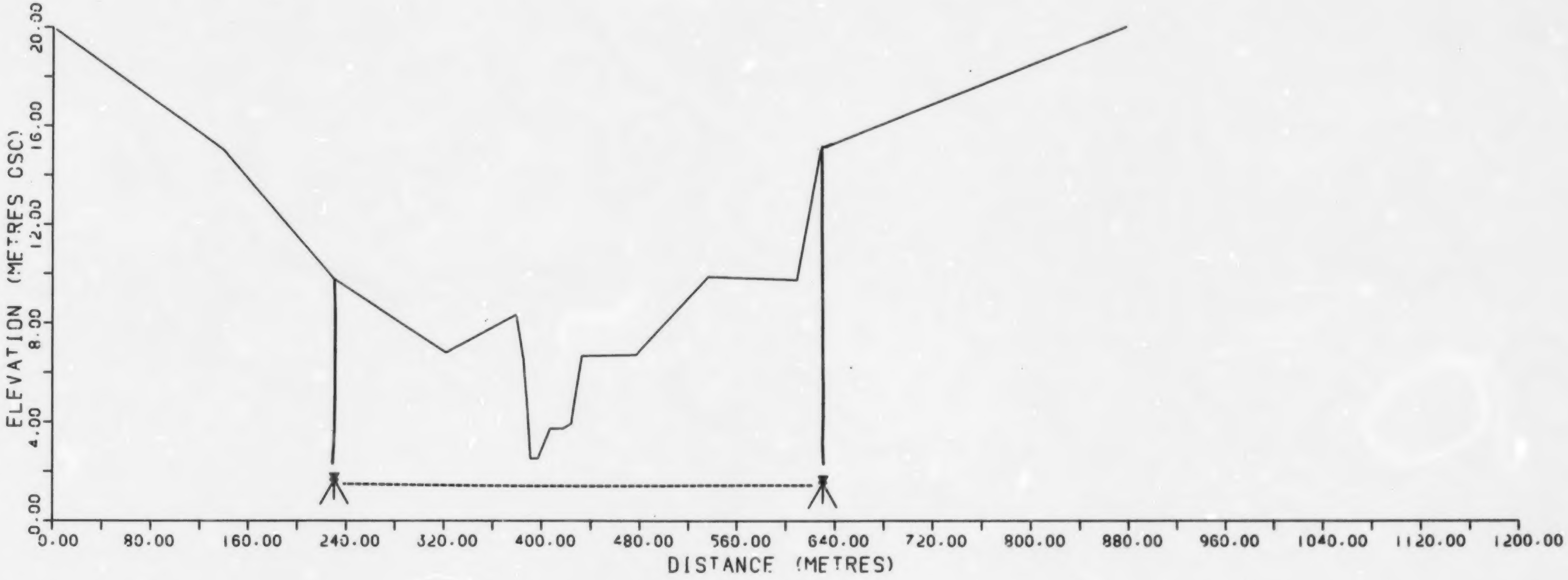


FIGURE A-22

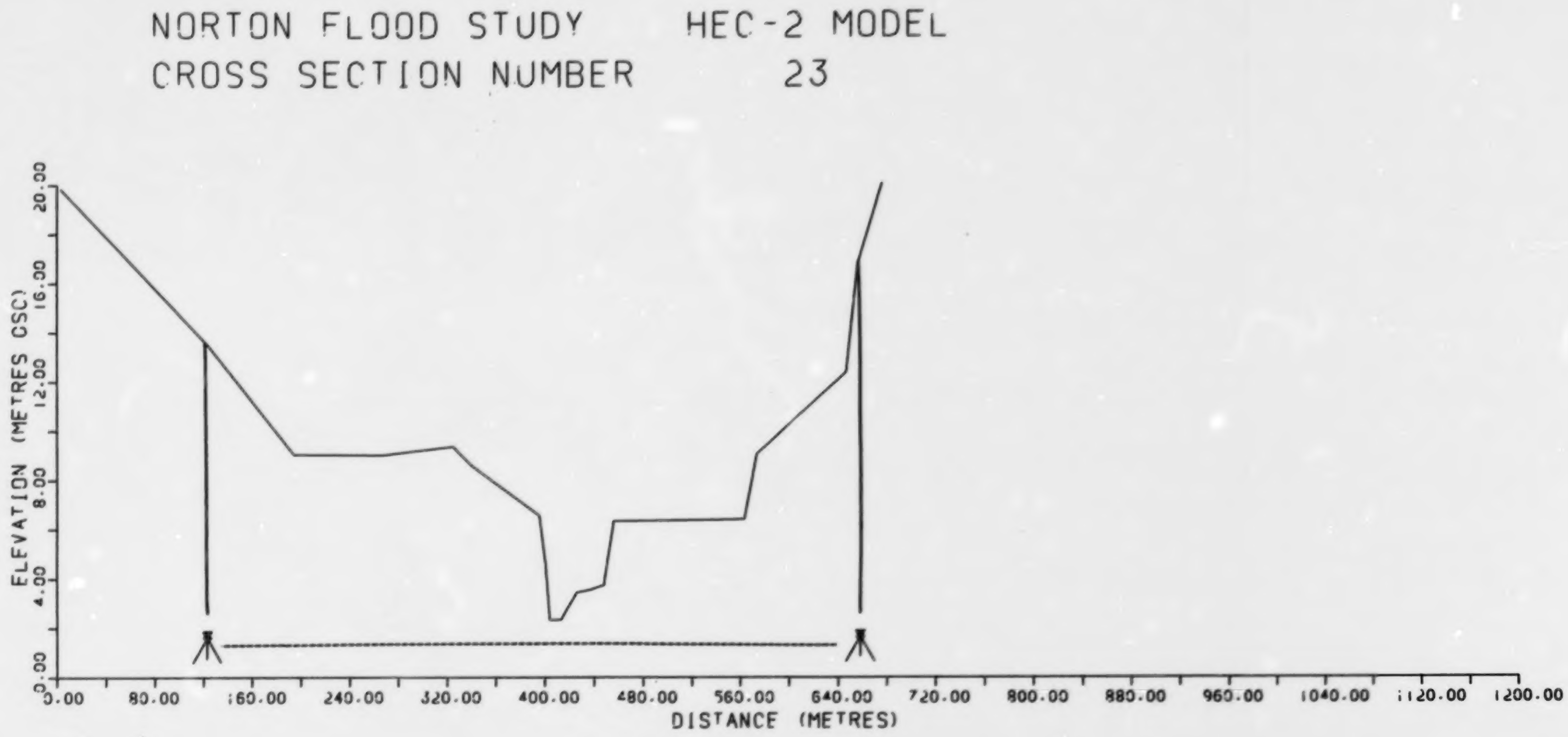

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 23
ELEVATION (METRES CSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE A-23

HEC-2 MODEL

CROSS SECTION NUMBER 25.1

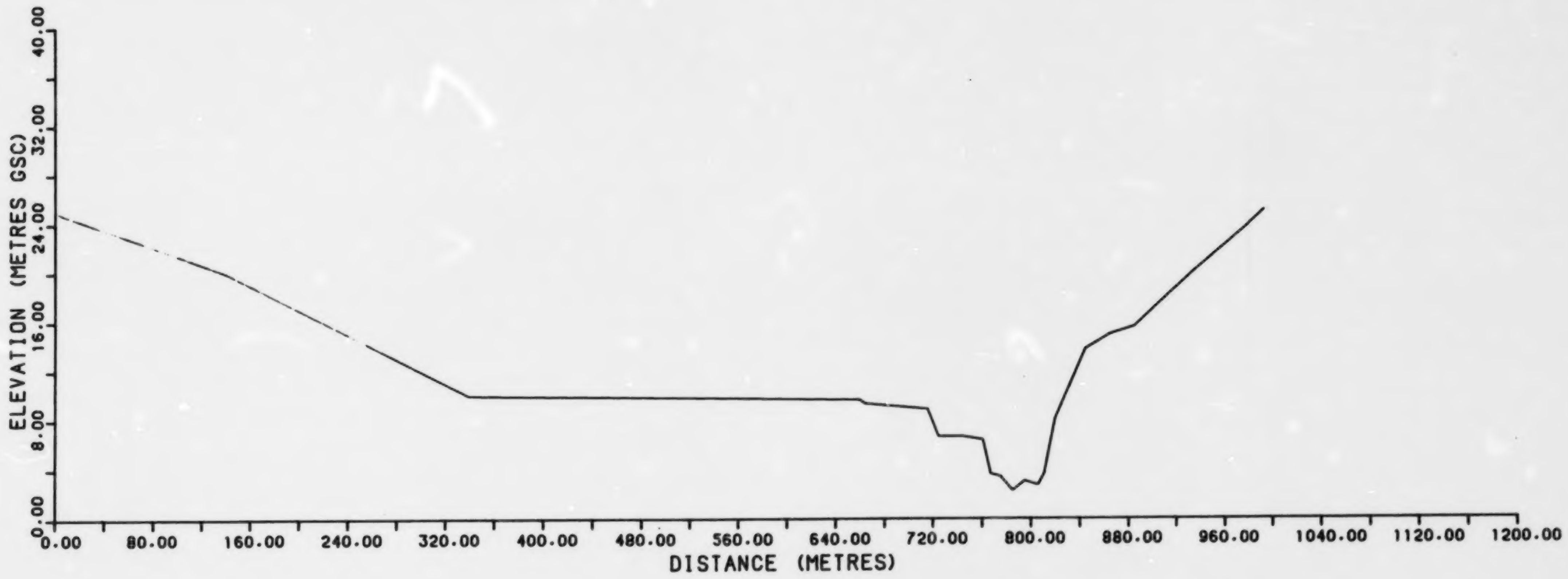


FIGURE A-24

HEC-2 MODEL

CROSS SECTION NUMBER 25.2

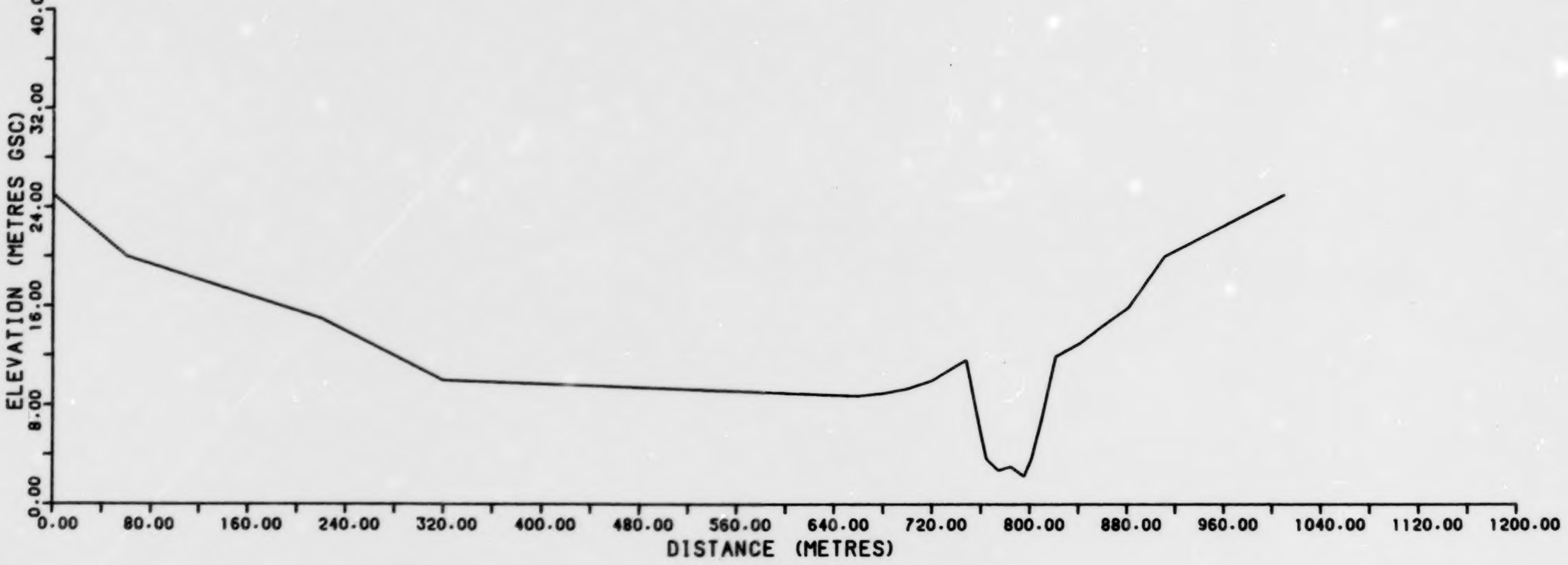


FIGURE A-25

HEC-2 MODEL

CROSS SECTION NUMBER 25.3

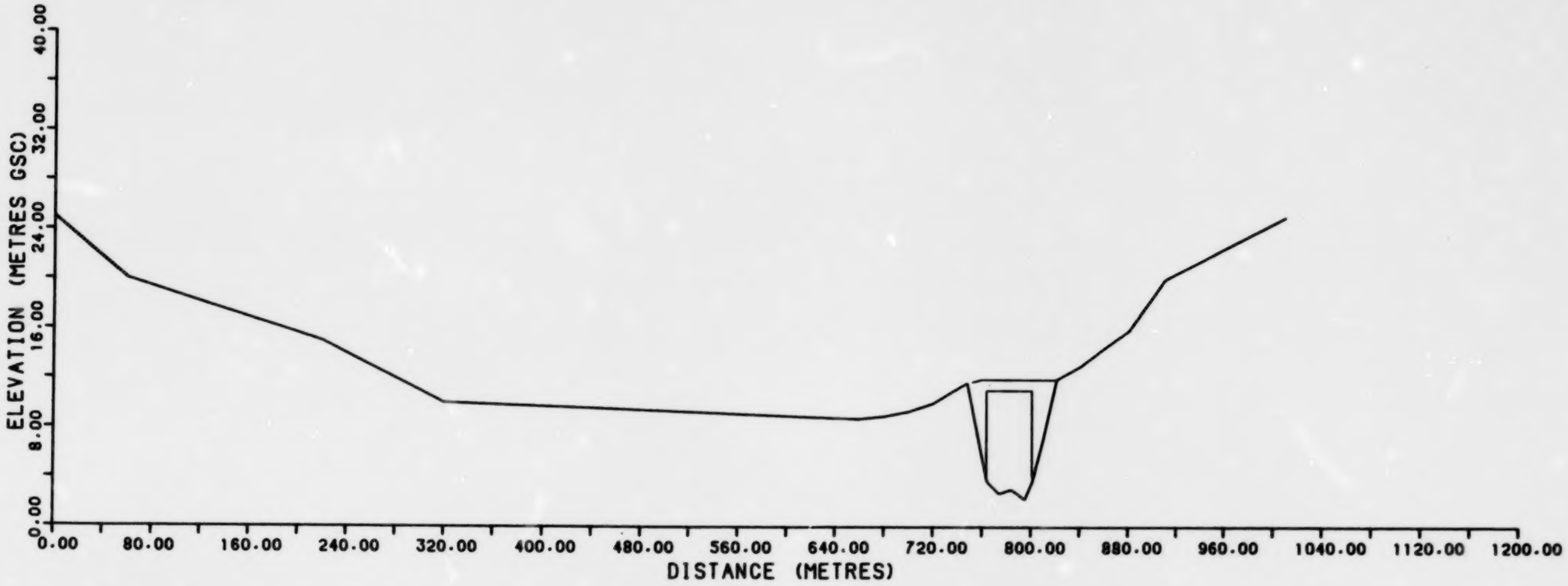


FIGURE A-26

HEC-2 MODEL

CROSS SECTION NUMBER 25.4

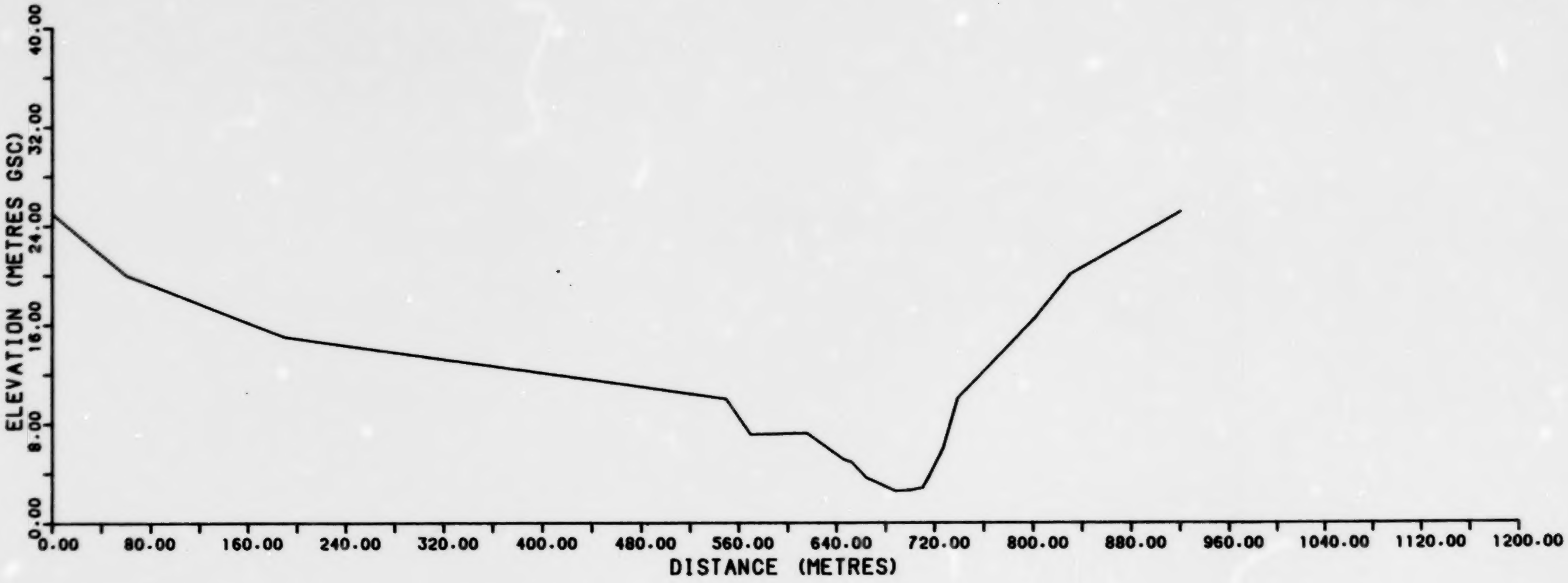


FIGURE A-27

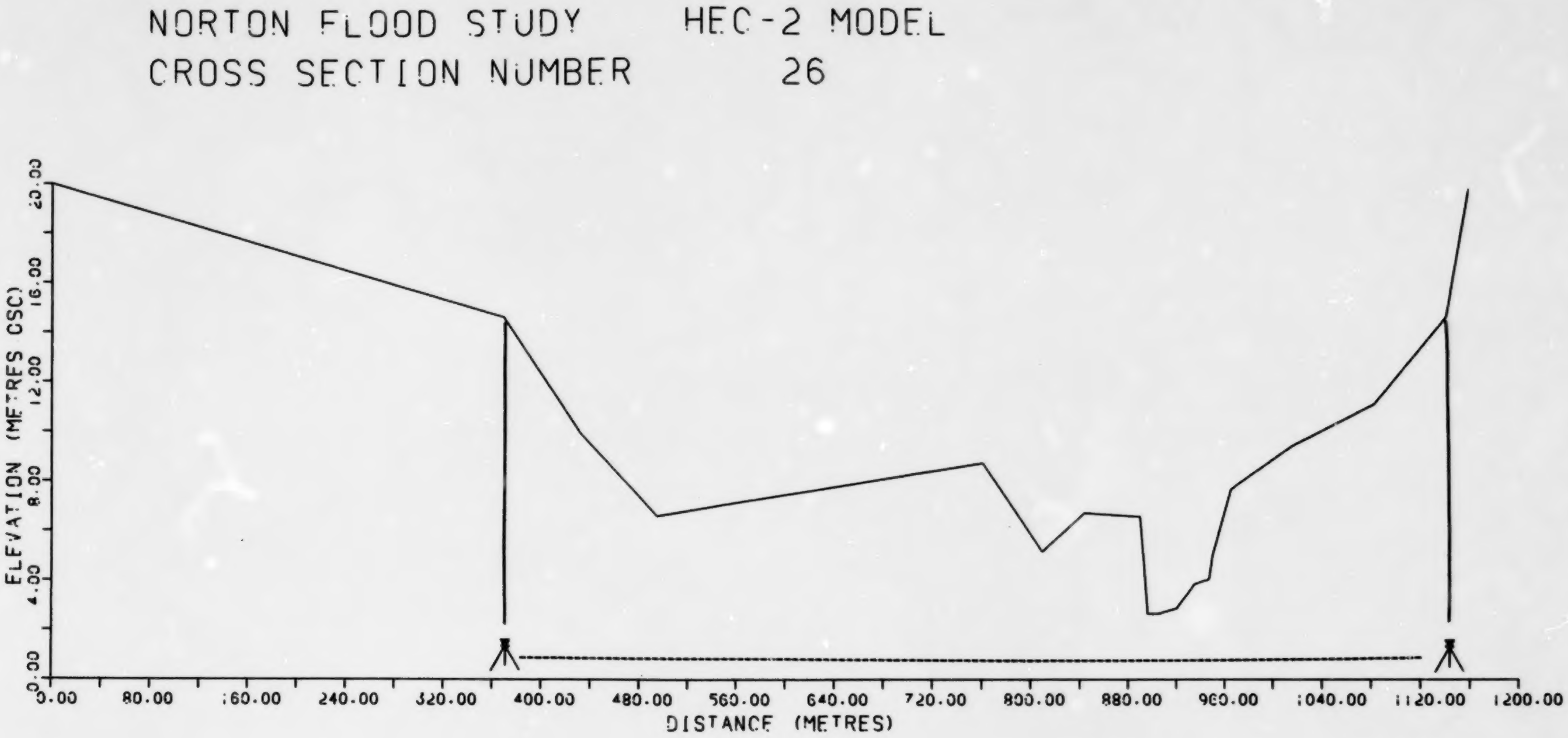

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 26
ELEVATION (METRES CSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE A-28

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 27

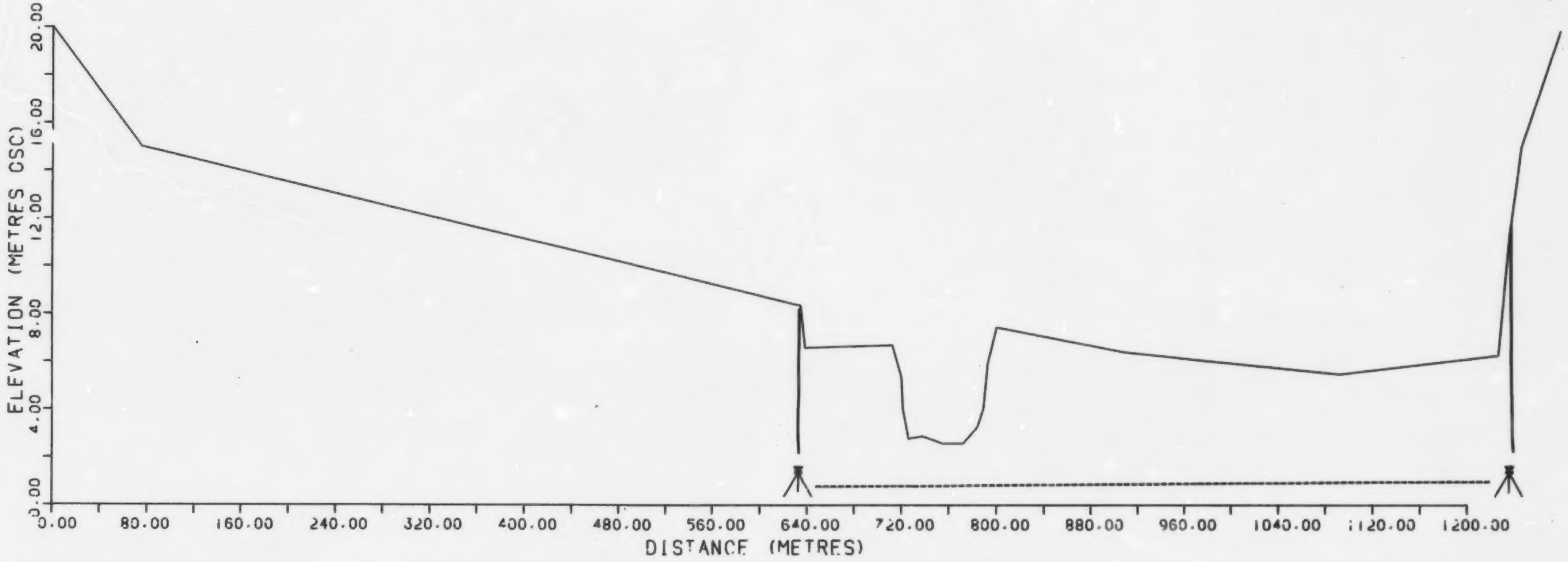


FIGURE A-29

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 28

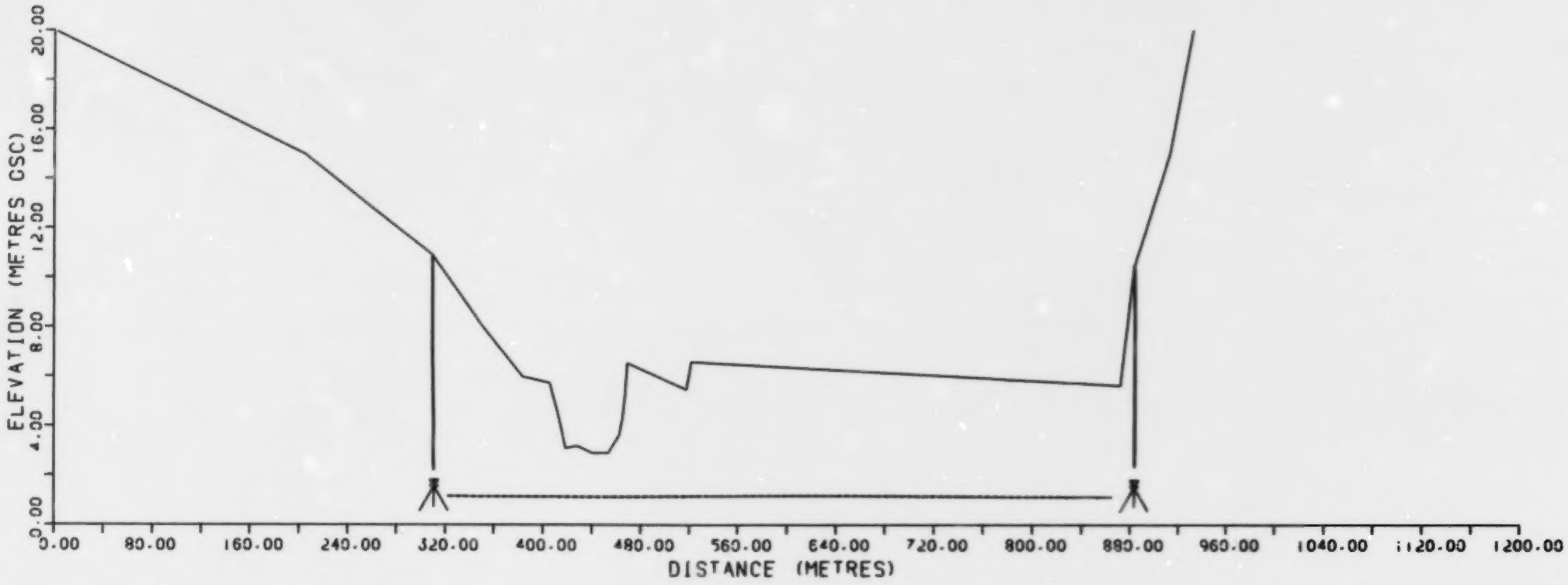


FIGURE A-30

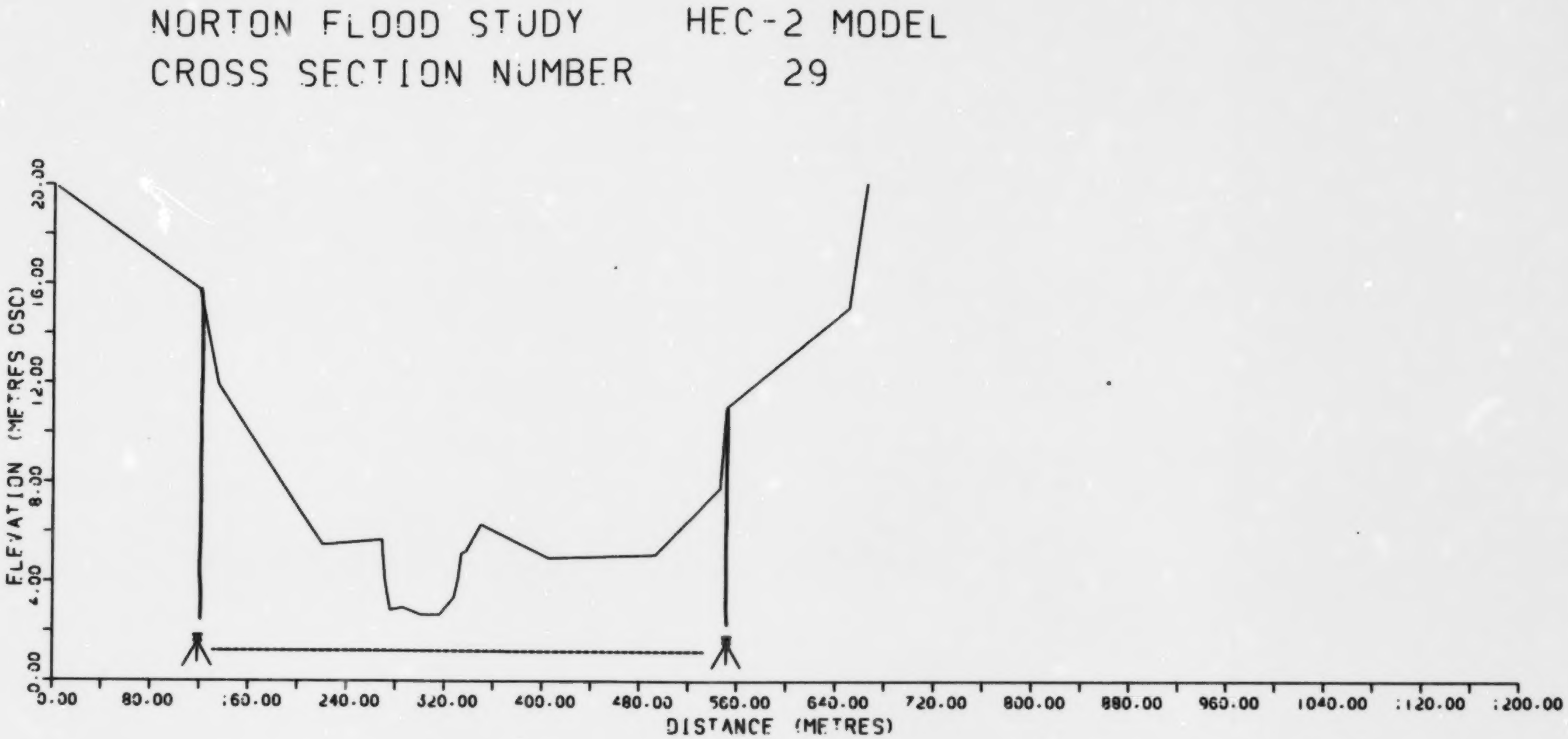

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 29

FIGURE A-31

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 30

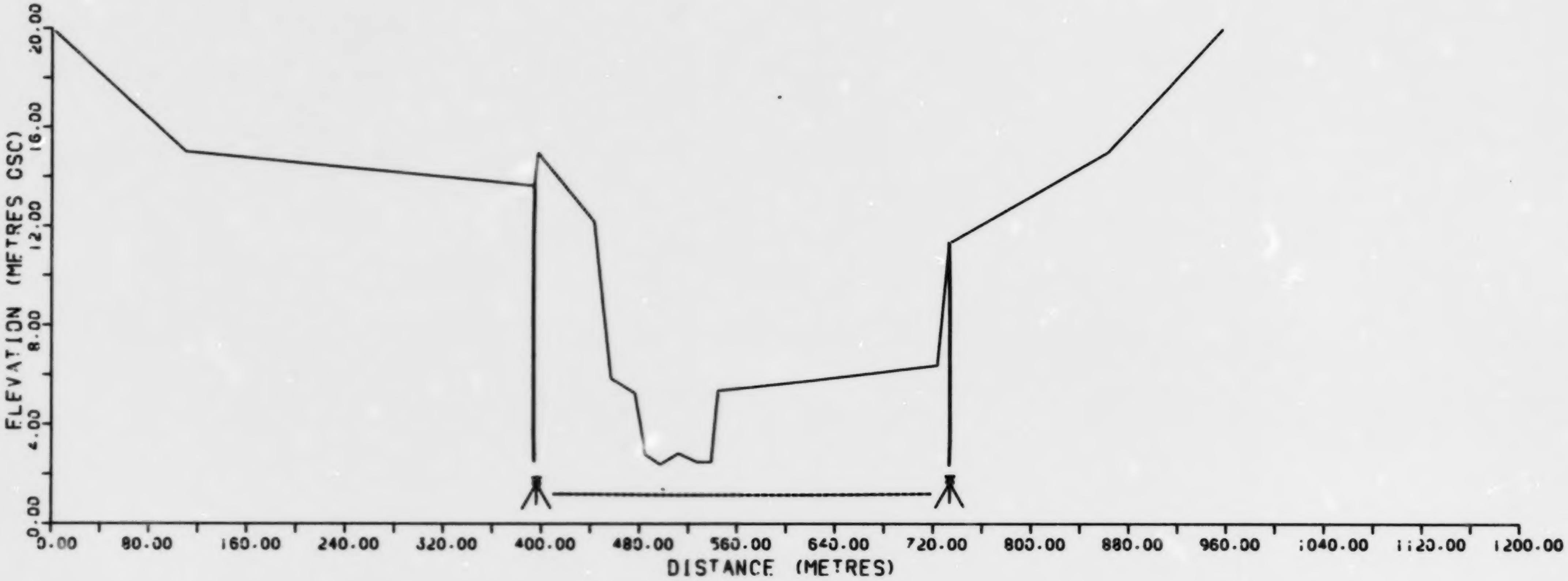


FIGURE A-32

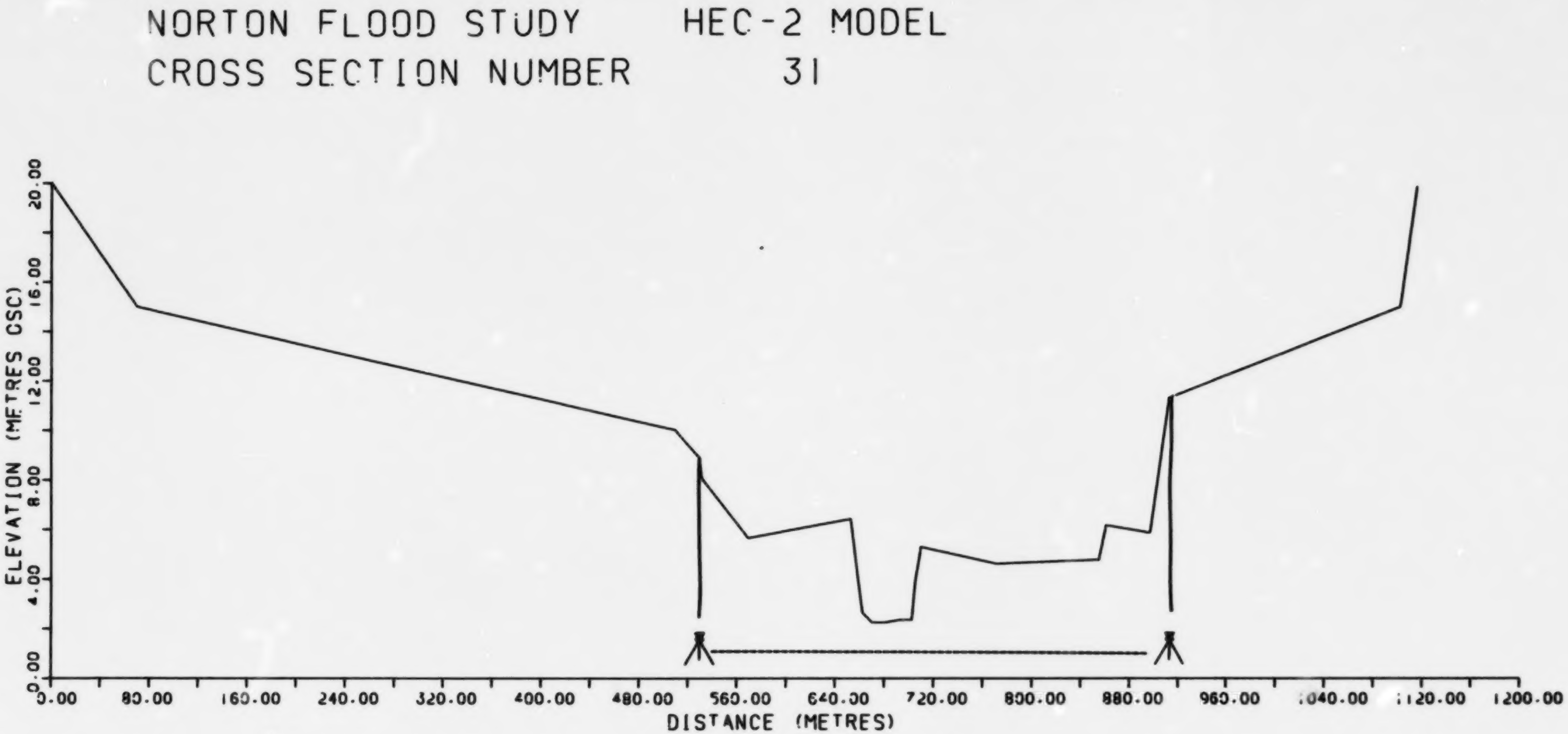

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 31
ELEVATION (METRES CSC)
0.00
4.00
8.00
12.00
16.00
20.00
DISTANCE (METRES)
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00

FIGURE A-33

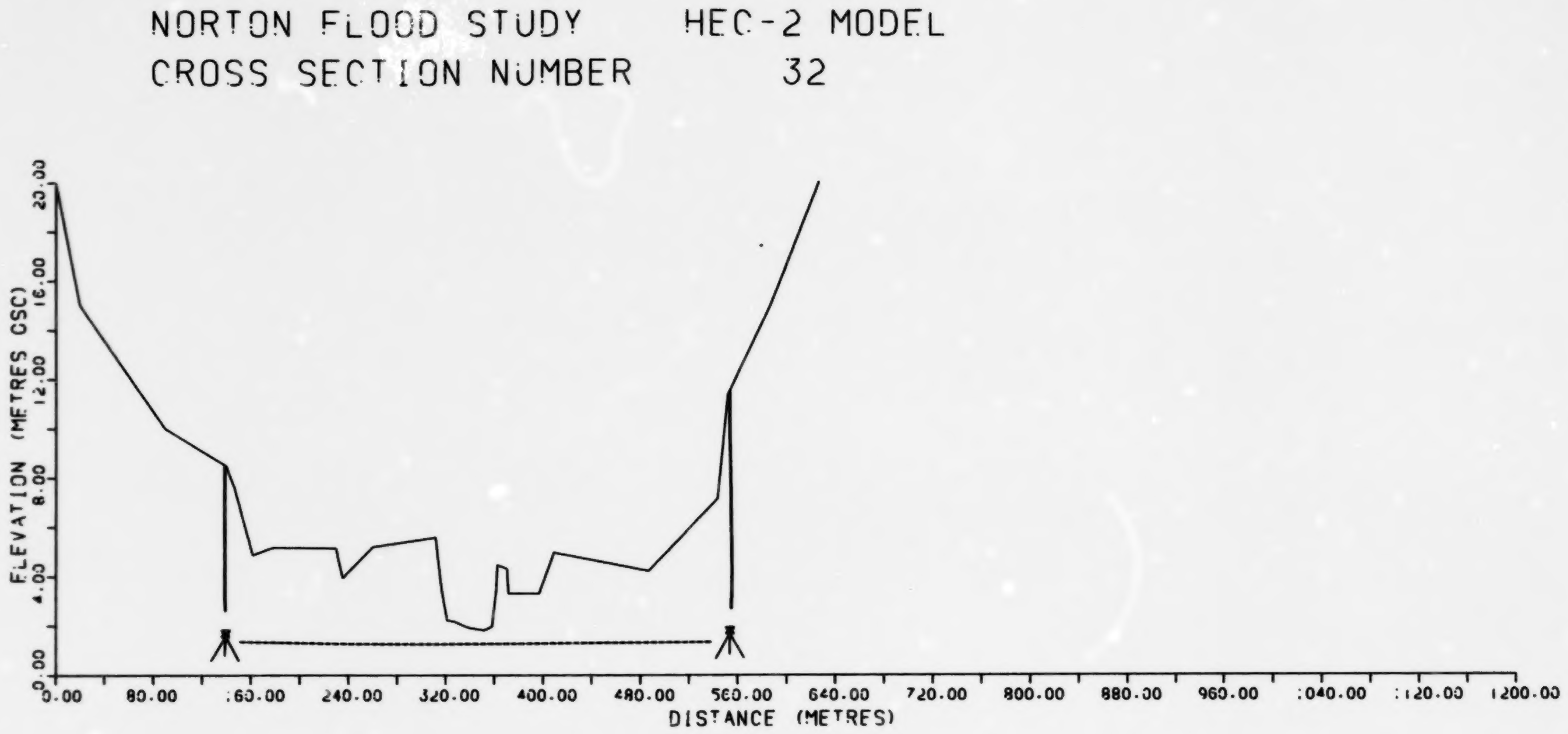

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 32
ELEVATION (METRES GSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE A-34

APPENDIX B

PLOTTED CROSS SECTIONS - DOWNSTREAM HYDRAULIC MODEL

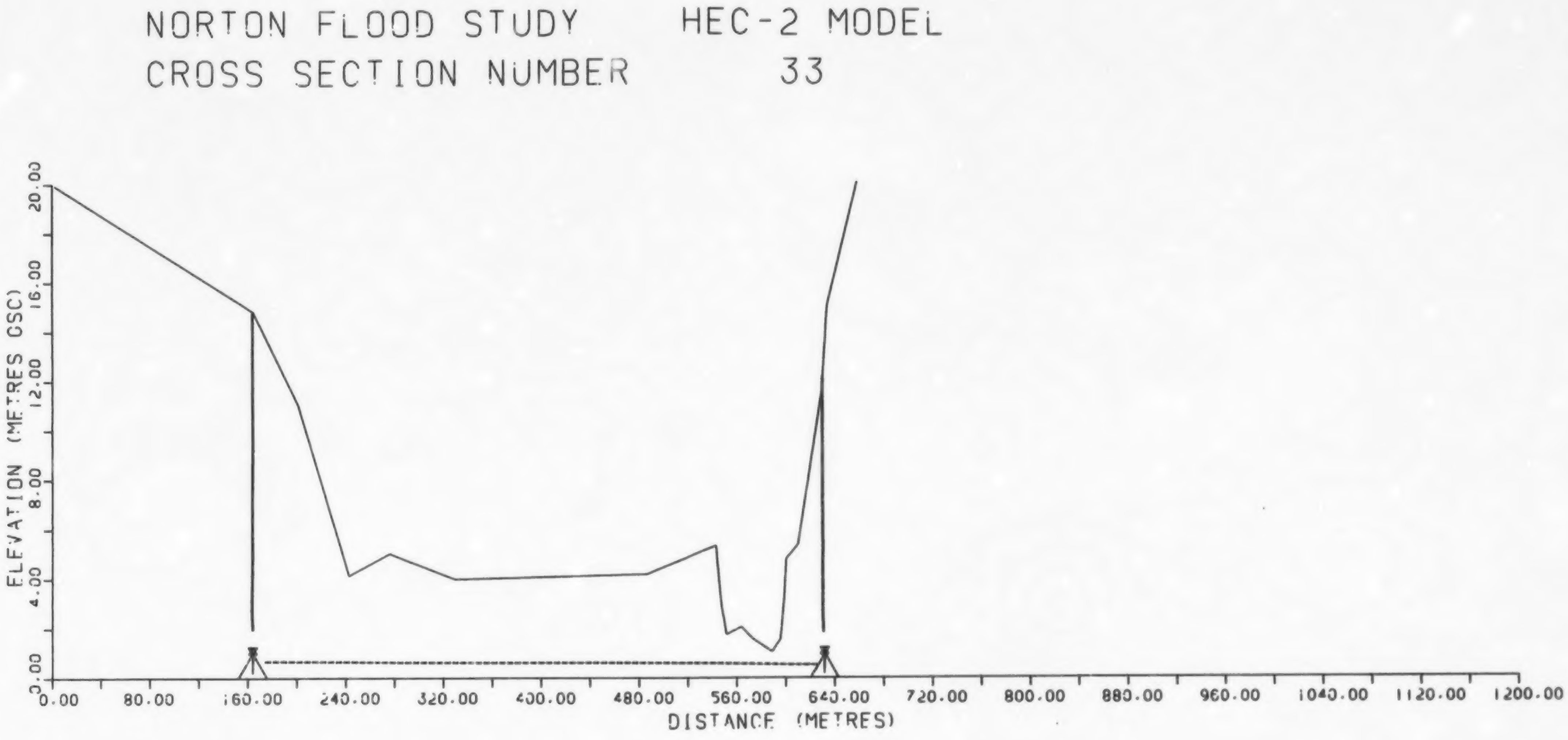

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 33
ELEVATION (METRES GSC)
0.00
4.00
8.00
12.00
16.00
20.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE B-1

NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 34

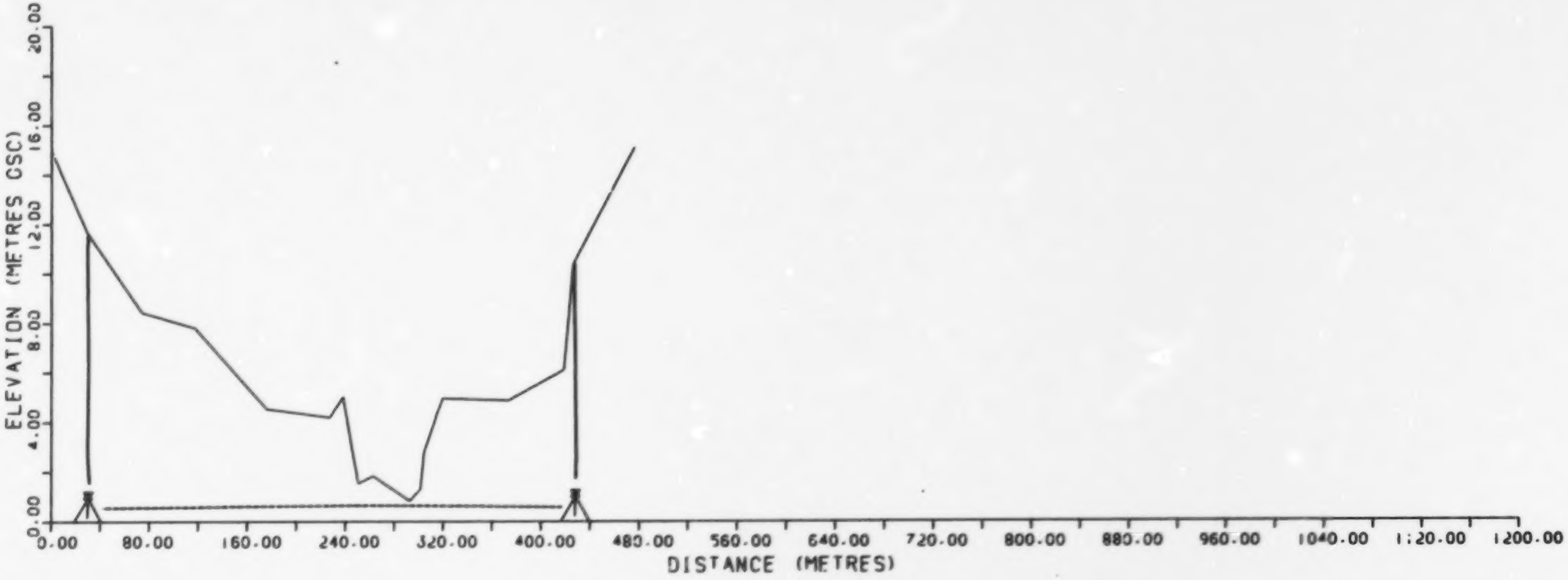
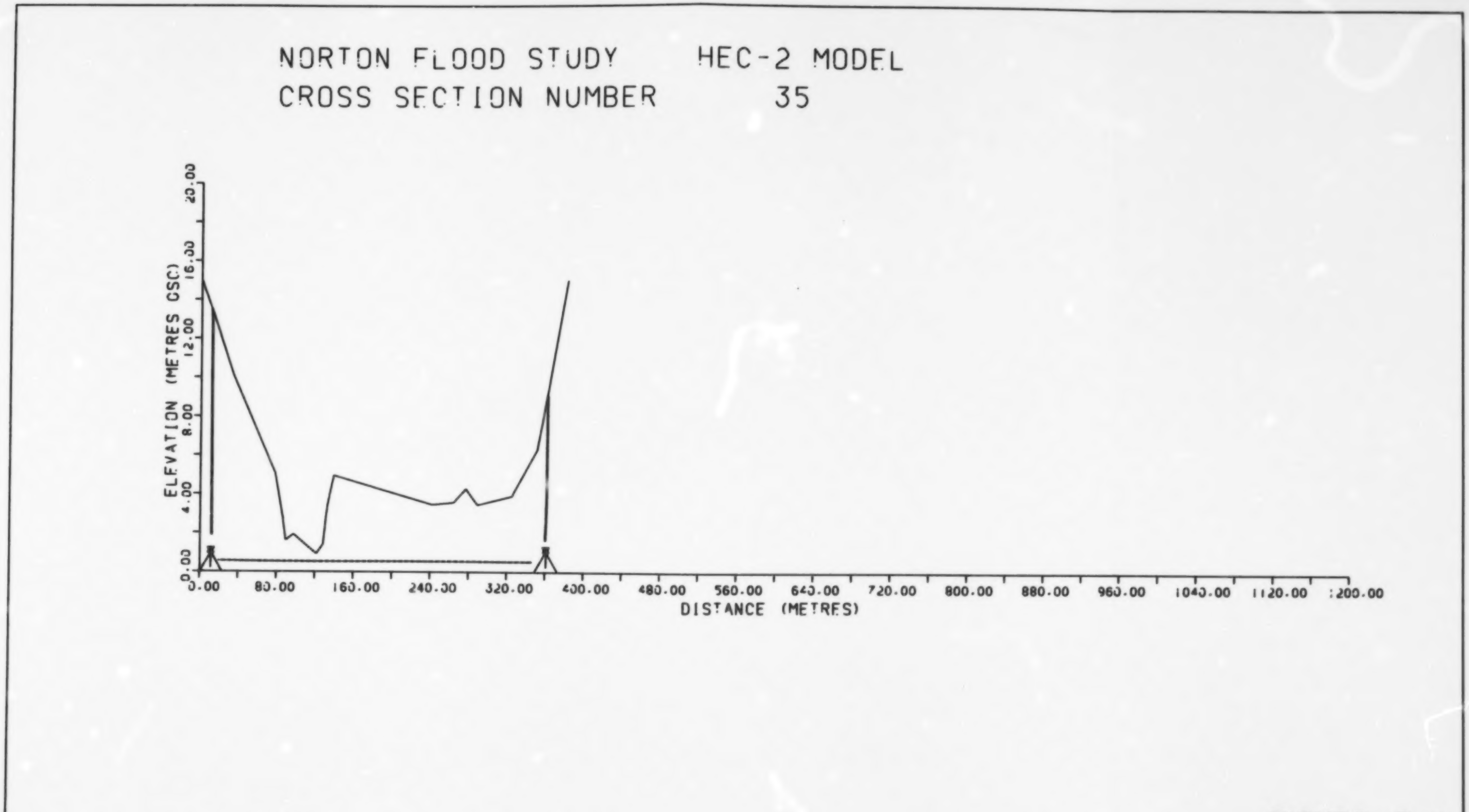


FIGURE B-2



NORTON FLOOD STUDY HEC-2 MODEL
CROSS SECTION NUMBER 35

FIGURE B-3

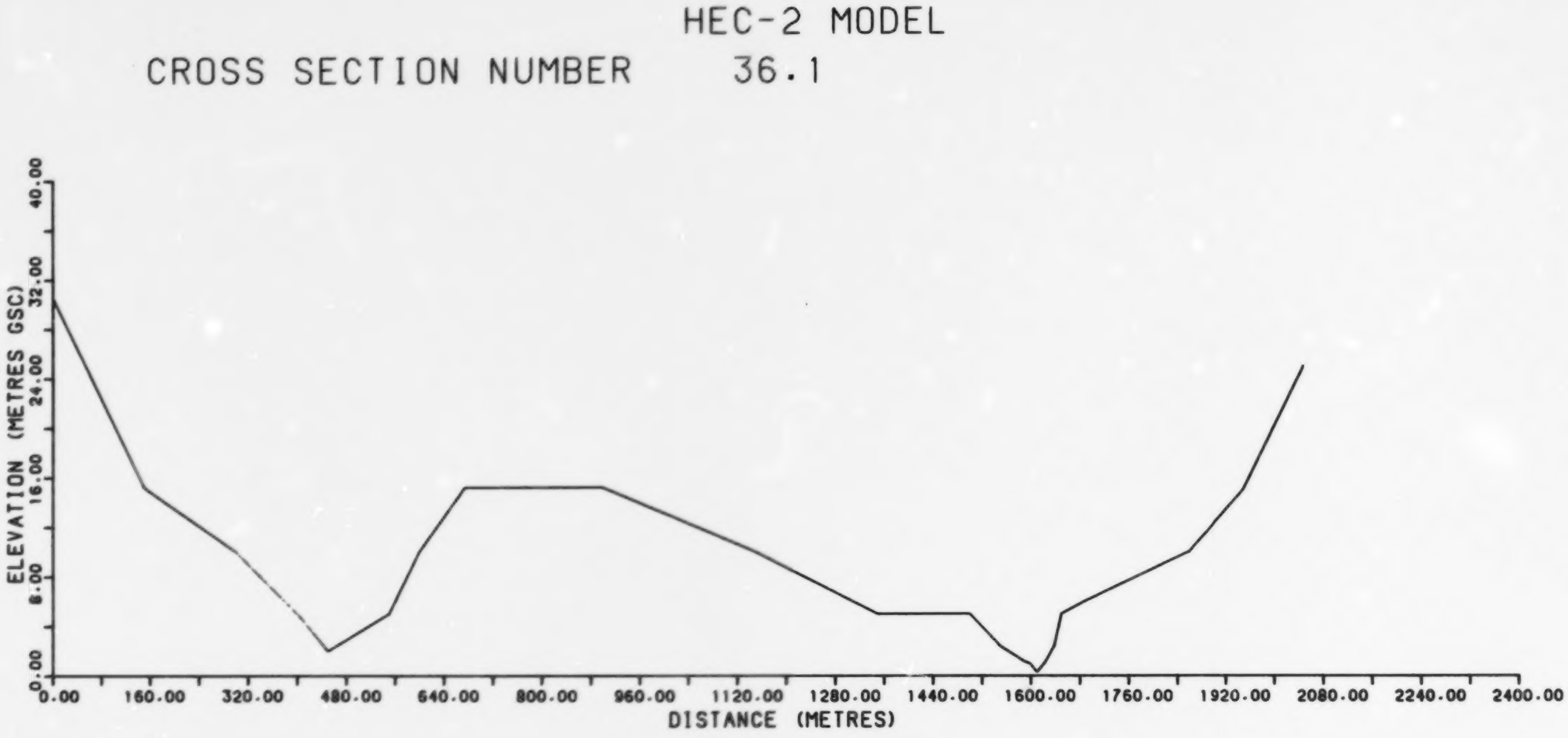

HEC-2 MODEL
CROSS SECTION NUMBER 36.1
ELEVATION (METRES GSC)
0.00
8.00
16.00
24.00
32.00
40.00
0.00
160.00
320.00
480.00
640.00
800.00
960.00
1120.00
1280.00
1440.00
1600.00
1760.00
1920.00
2080.00
2240.00
2400.00
DISTANCE (METRES)

FIGURE B-4

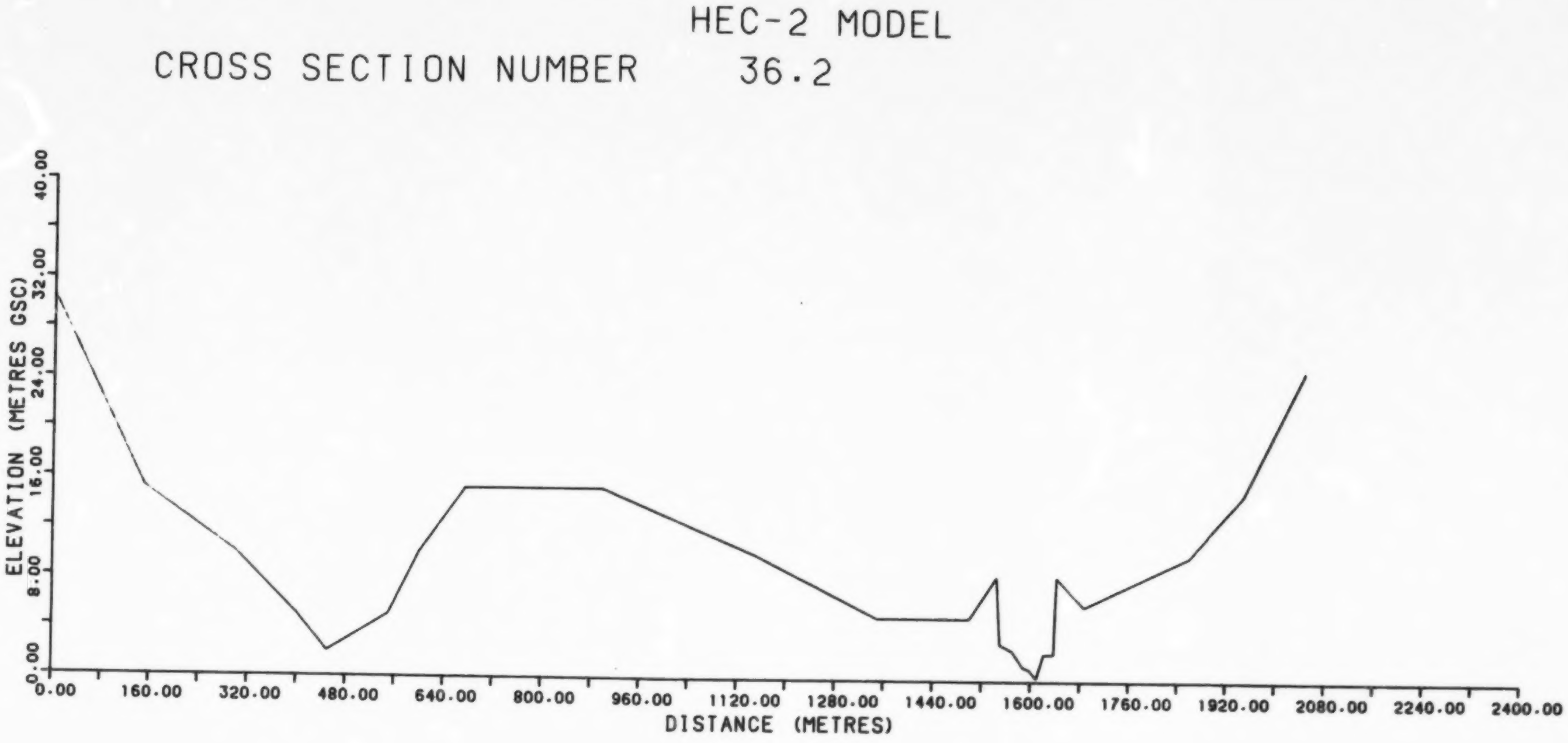

HEC-2 MODEL
CROSS SECTION NUMBER 36.2
ELEVATION (METRES GSC)
0.00
8.00
16.00
24.00
32.00
40.00
0.00
160.00
320.00
480.00
640.00
800.00
960.00
1120.00
1280.00
1440.00
1600.00
1760.00
1920.00
2080.00
2240.00
2400.00
DISTANCE (METRES)

FIGURE B-5

HEC-2 MODEL

CROSS SECTION NUMBER 36.3

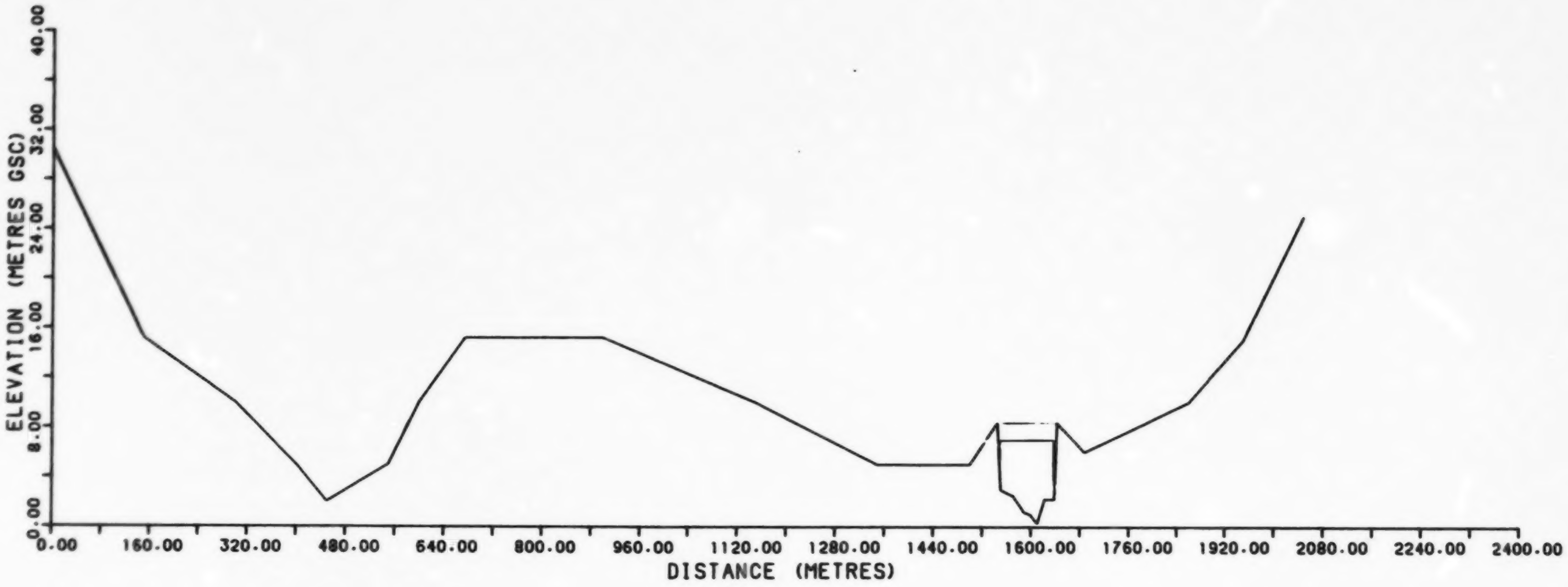
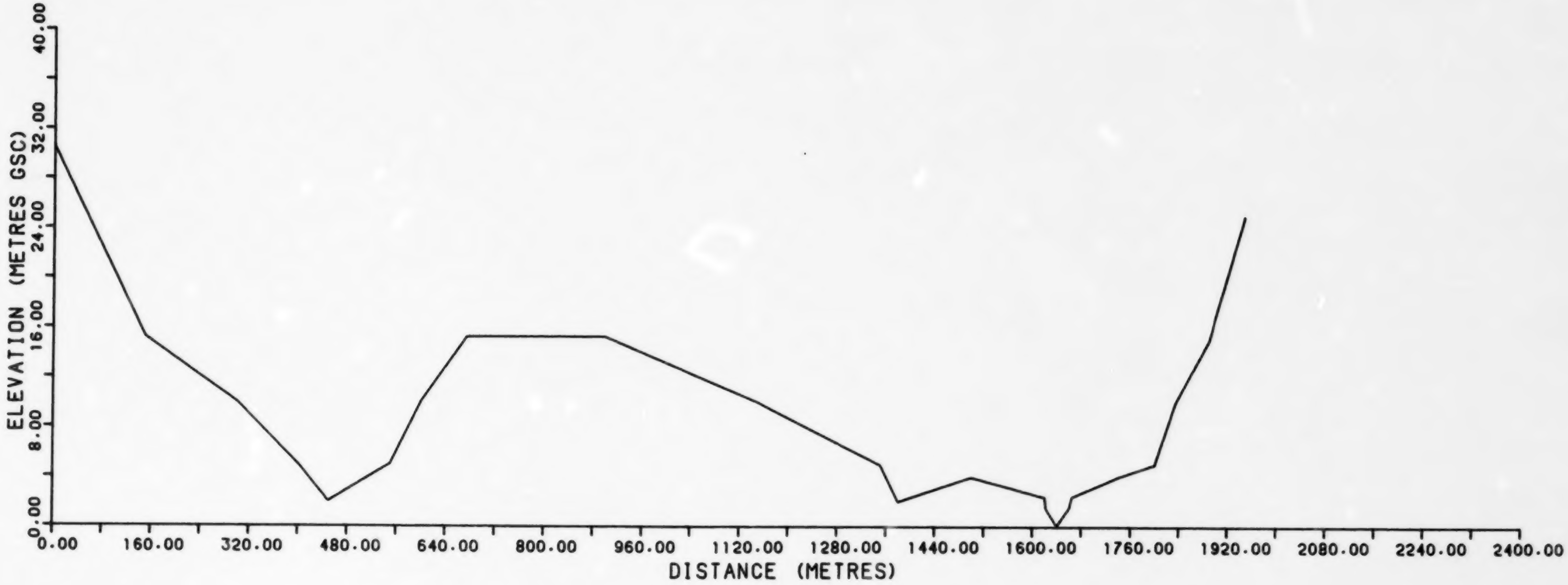


FIGURE B-6



HEC-2 MODEL

CROSS SECTION NUMBER 36.4

FIGURE B-7

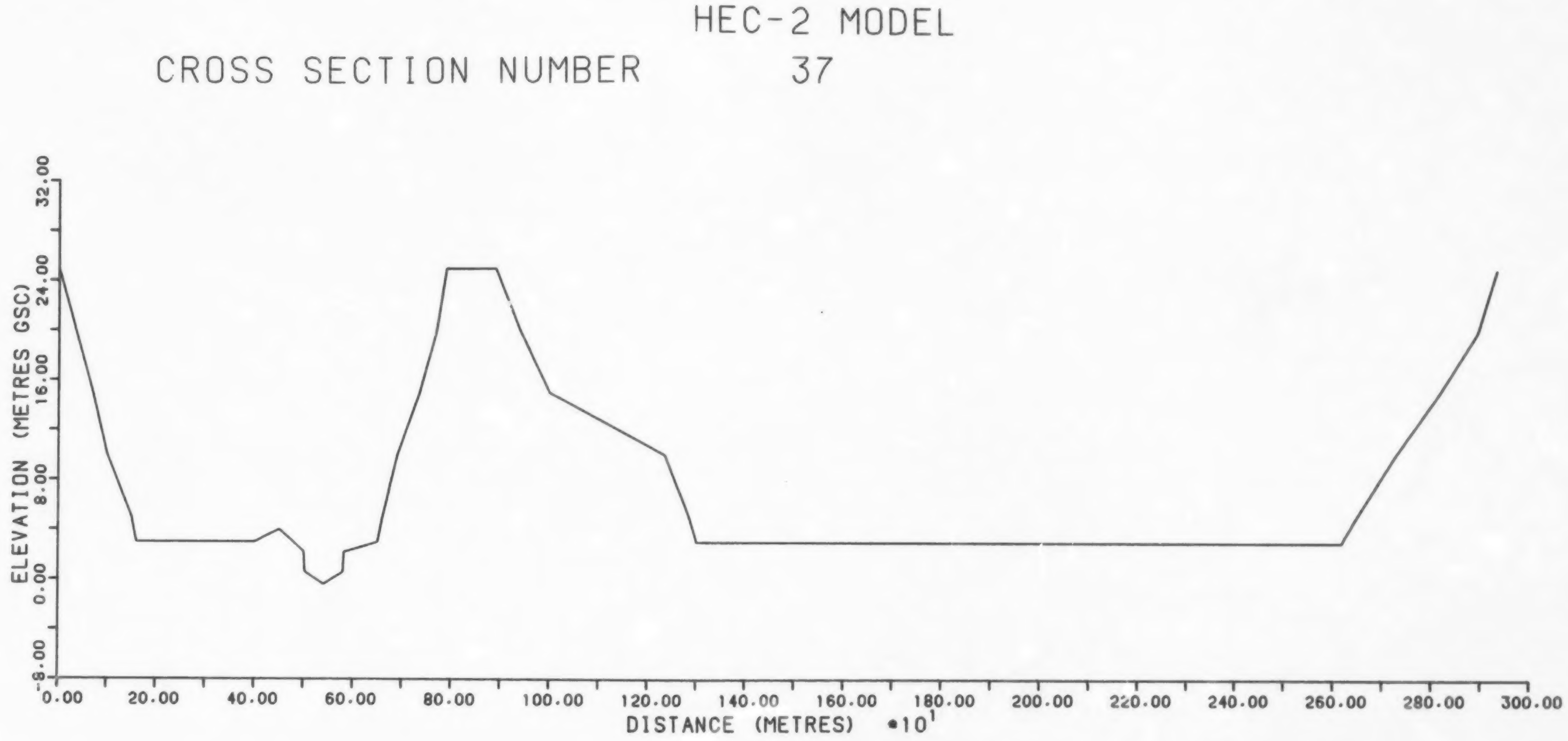

HEC-2 MODEL
CROSS SECTION NUMBER 37
ELEVATION (METRES GSC)
32.00
24.00
16.00
8.00
0.00
-8.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00
240.00
260.00
280.00
300.00
DISTANCE (METRES) *10¹

FIGURE B-8

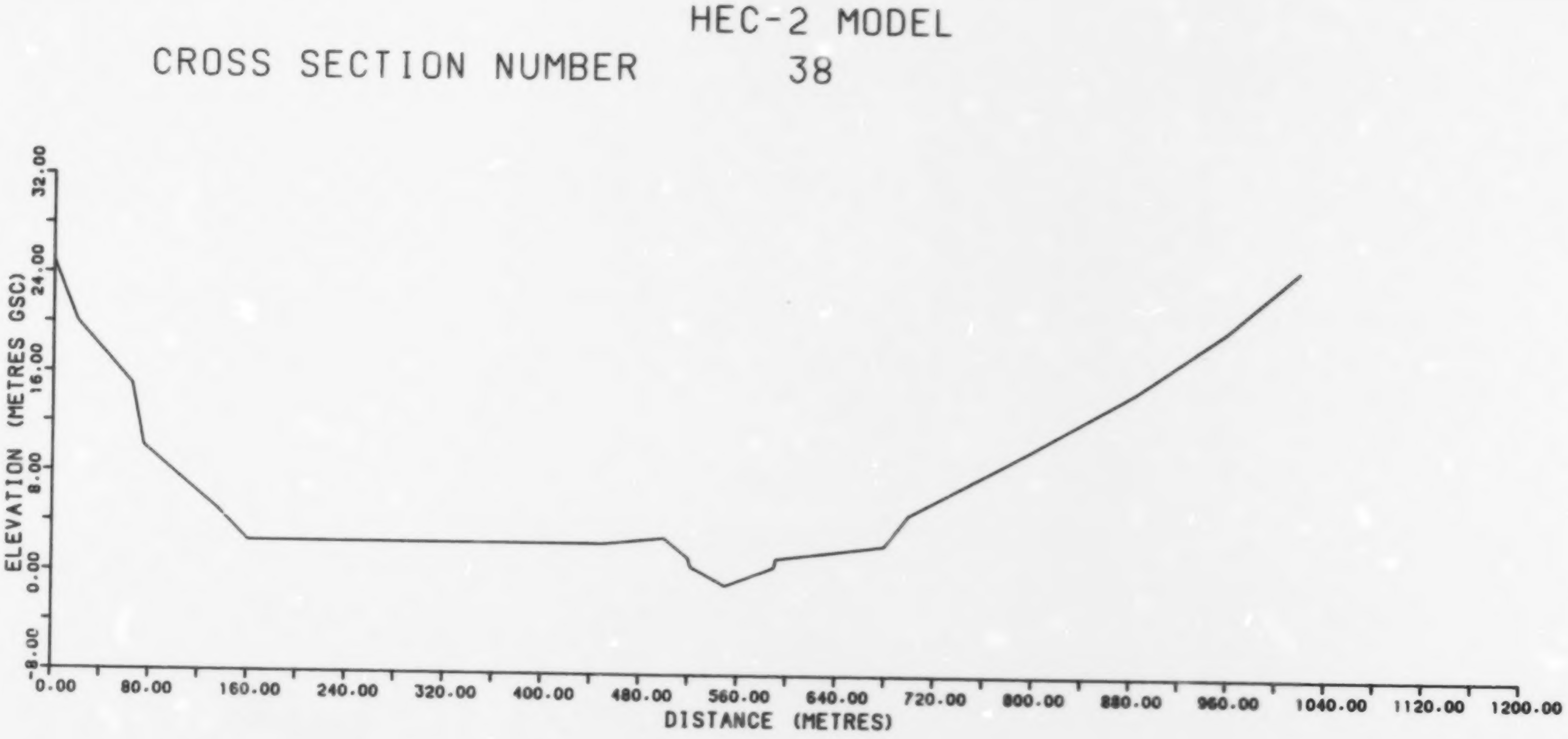

HEC-2 MODEL
CROSS SECTION NUMBER 38
ELEVATION (METRES GSC)
-8.00
0.00
8.00
16.00
24.00
32.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE B-9

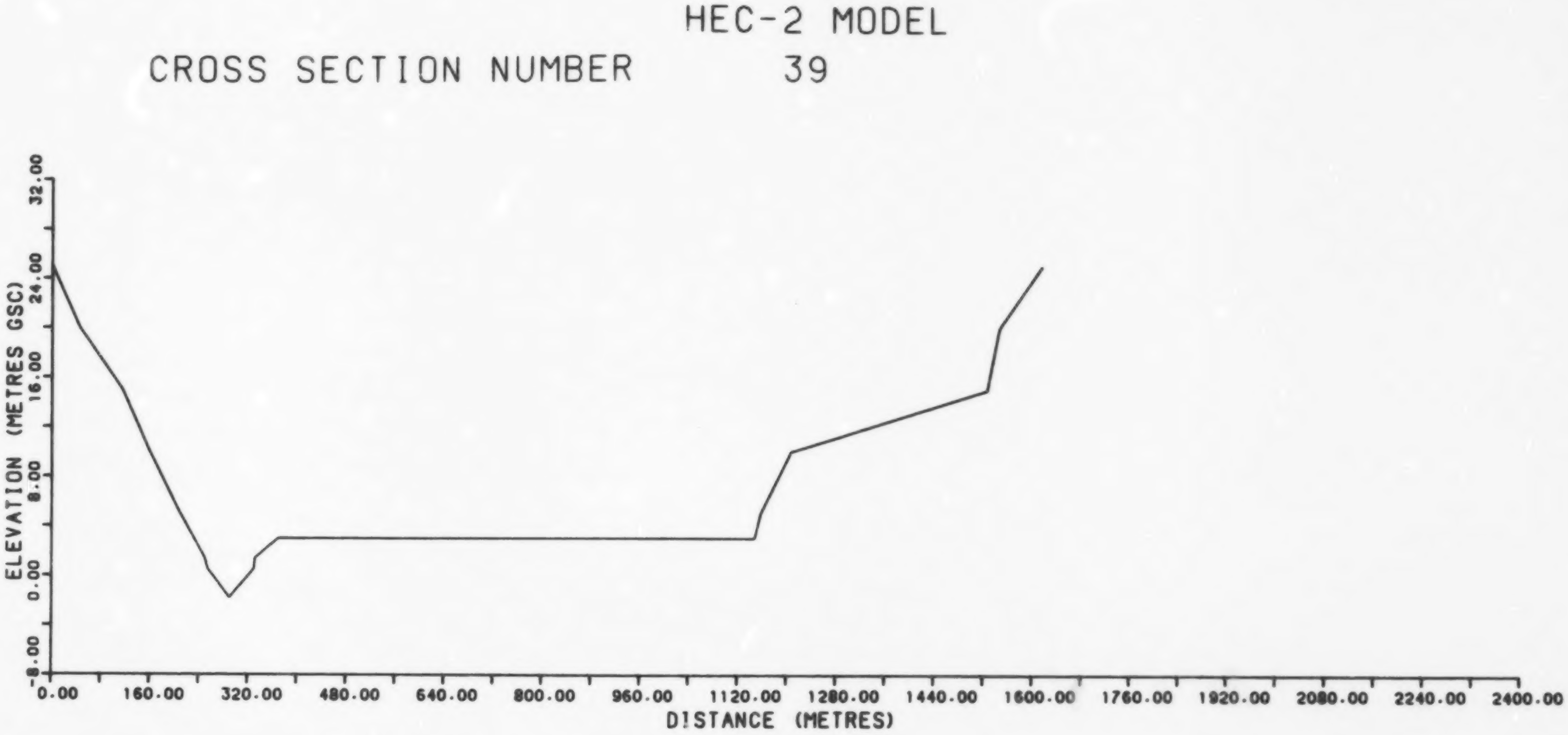

HEC-2 MODEL
CROSS SECTION NUMBER 39
ELEVATION (METRES GSC)
-8.00
0.00
8.00
16.00
24.00
32.00
0.00
160.00
320.00
480.00
640.00
800.00
960.00
1120.00
1280.00
1440.00
1600.00
1760.00
1920.00
2080.00
2240.00
2400.00
DISTANCE (METRES)

FIGURE B-10

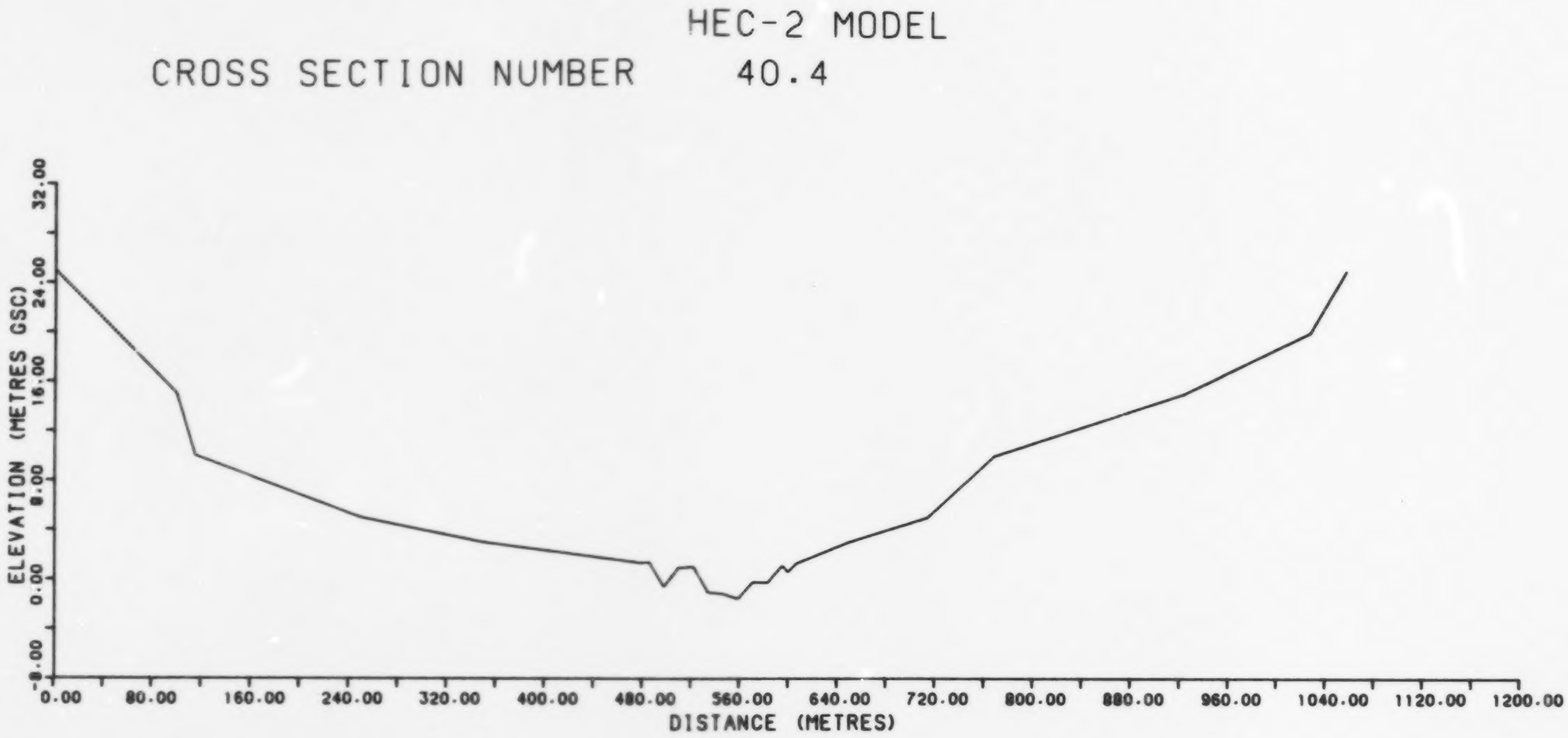

HEC-2 MODEL
CROSS SECTION NUMBER 40.4
ELEVATION (METRES GSC)
32.00
24.00
16.00
8.00
0.00
-8.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE B-11

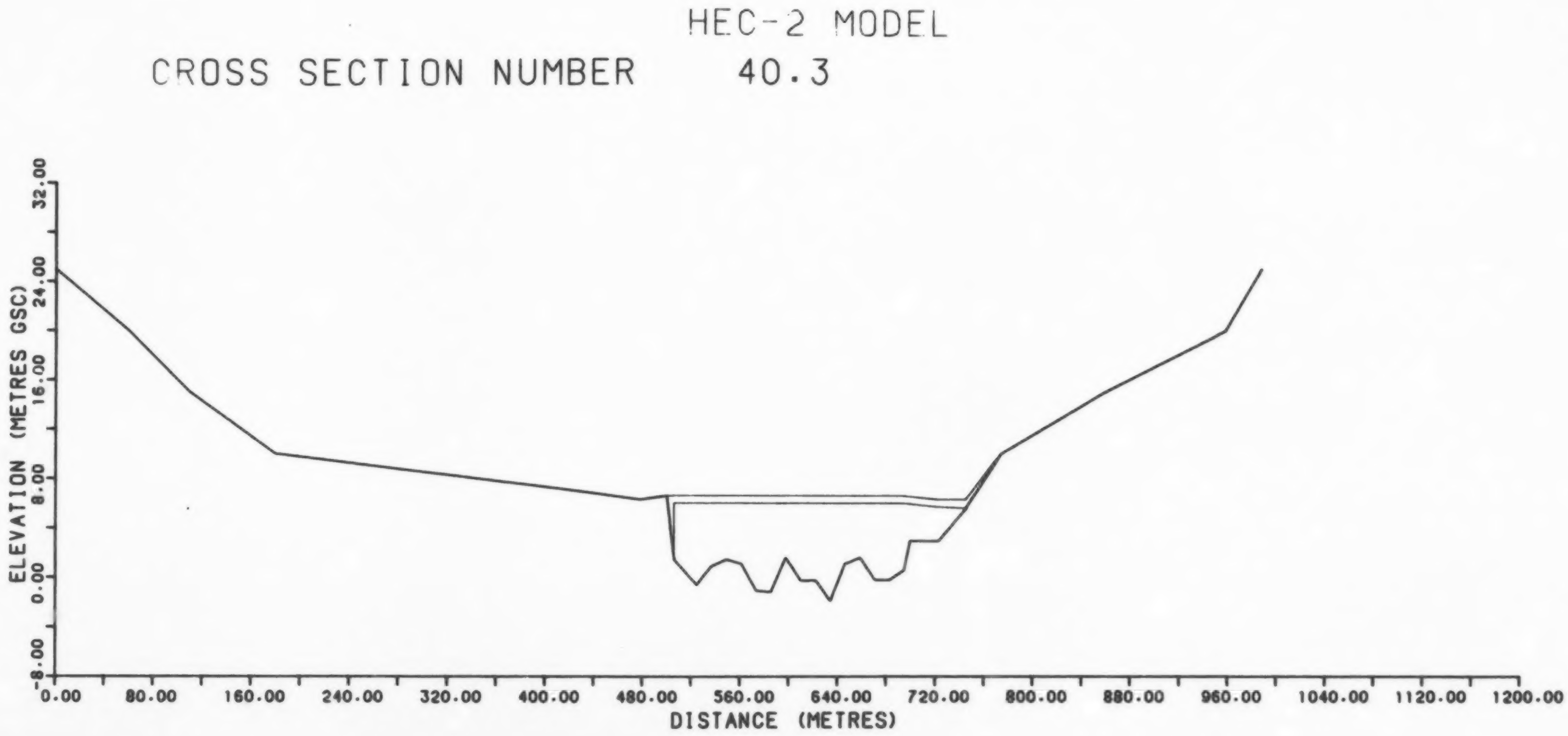

HEC-2 MODEL
CROSS SECTION NUMBER 40.3
ELEVATION (METRES GSC)
32.00
24.00
16.00
8.00
0.00
-8.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE B-12

HEC-2 MODEL

CROSS SECTION NUMBER 40.2

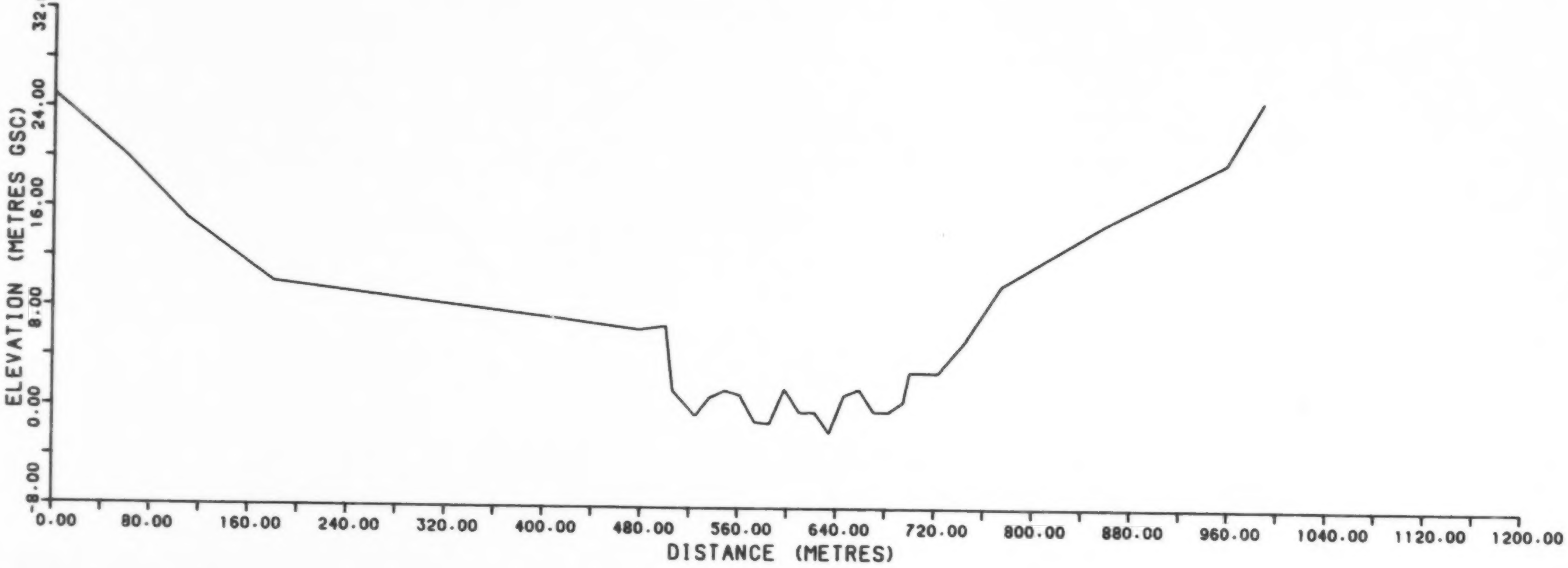


FIGURE B-13

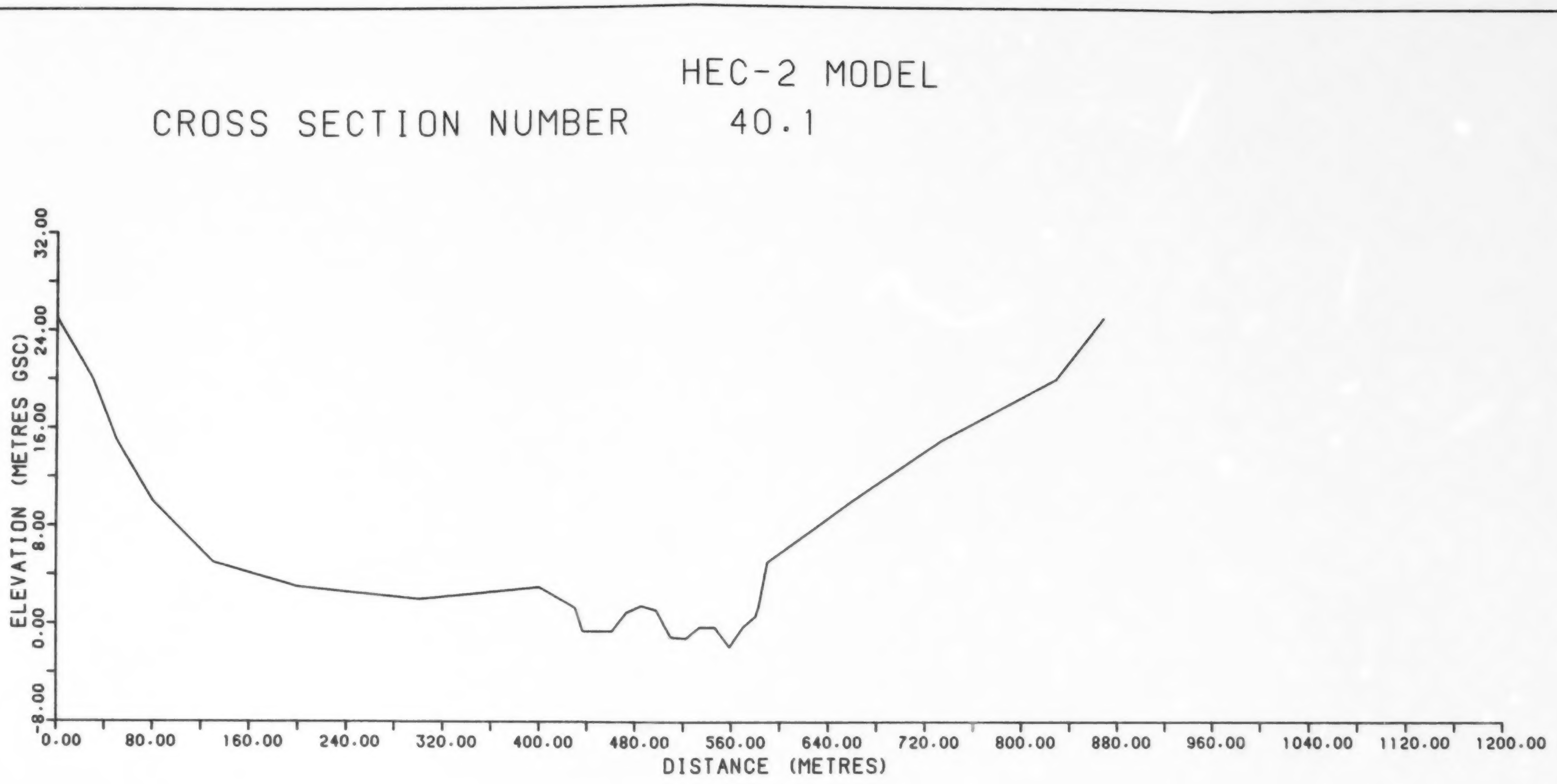

HEC-2 MODEL
CROSS SECTION NUMBER 40.1
ELEVATION (METRES GSC)
32.00
24.00
16.00
8.00
0.00
-8.00
0.00
80.00
160.00
240.00
320.00
400.00
480.00
560.00
640.00
720.00
800.00
880.00
960.00
1040.00
1120.00
1200.00
DISTANCE (METRES)

FIGURE B-14

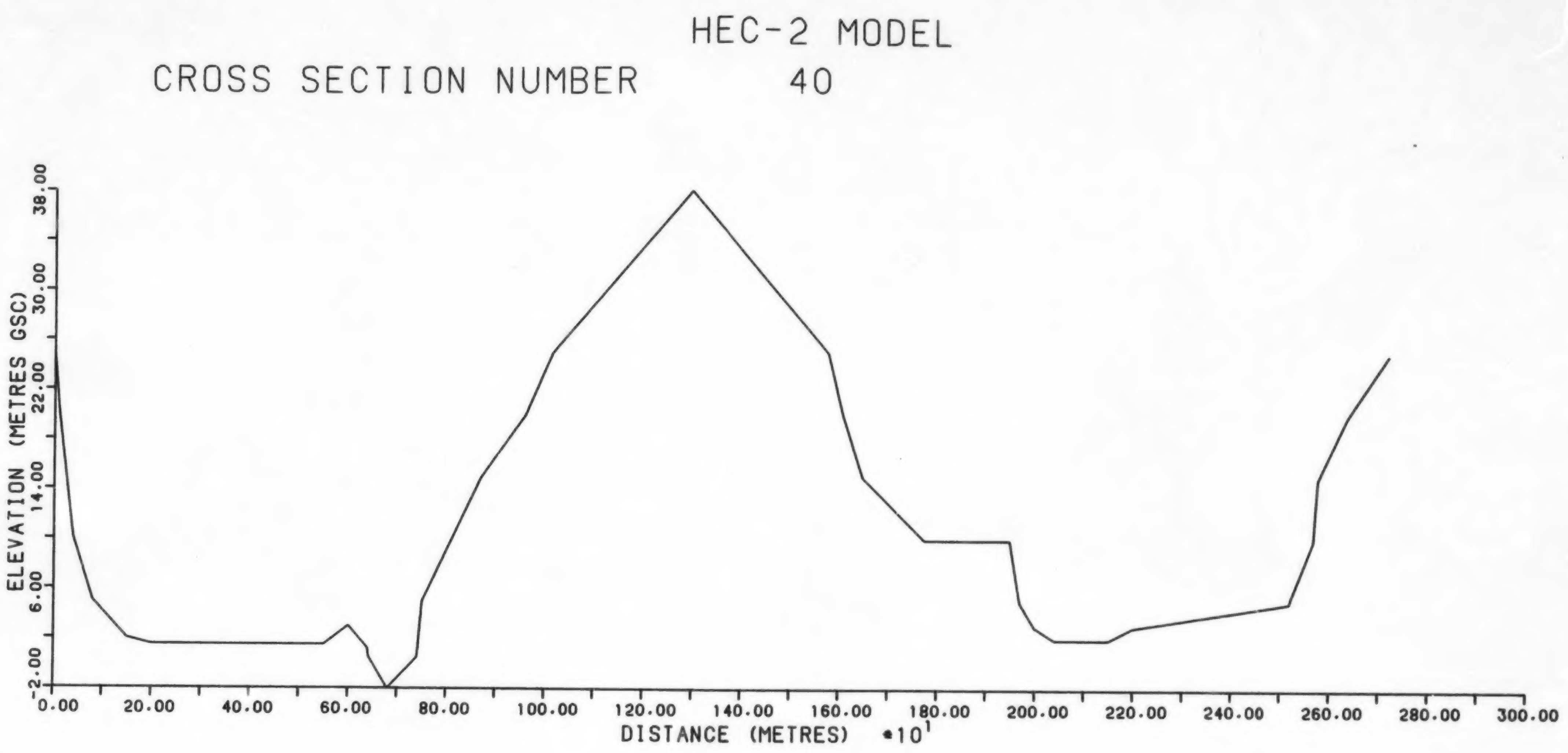

HEC-2 MODEL
CROSS SECTION NUMBER 40
ELEVATION (METRES GSC)
38.00
30.00
22.00
14.00
6.00
-2.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
140.00
160.00
180.00
200.00
220.00
240.00
260.00
280.00
300.00
DISTANCE (METRES) *10¹

FIGURE B-15

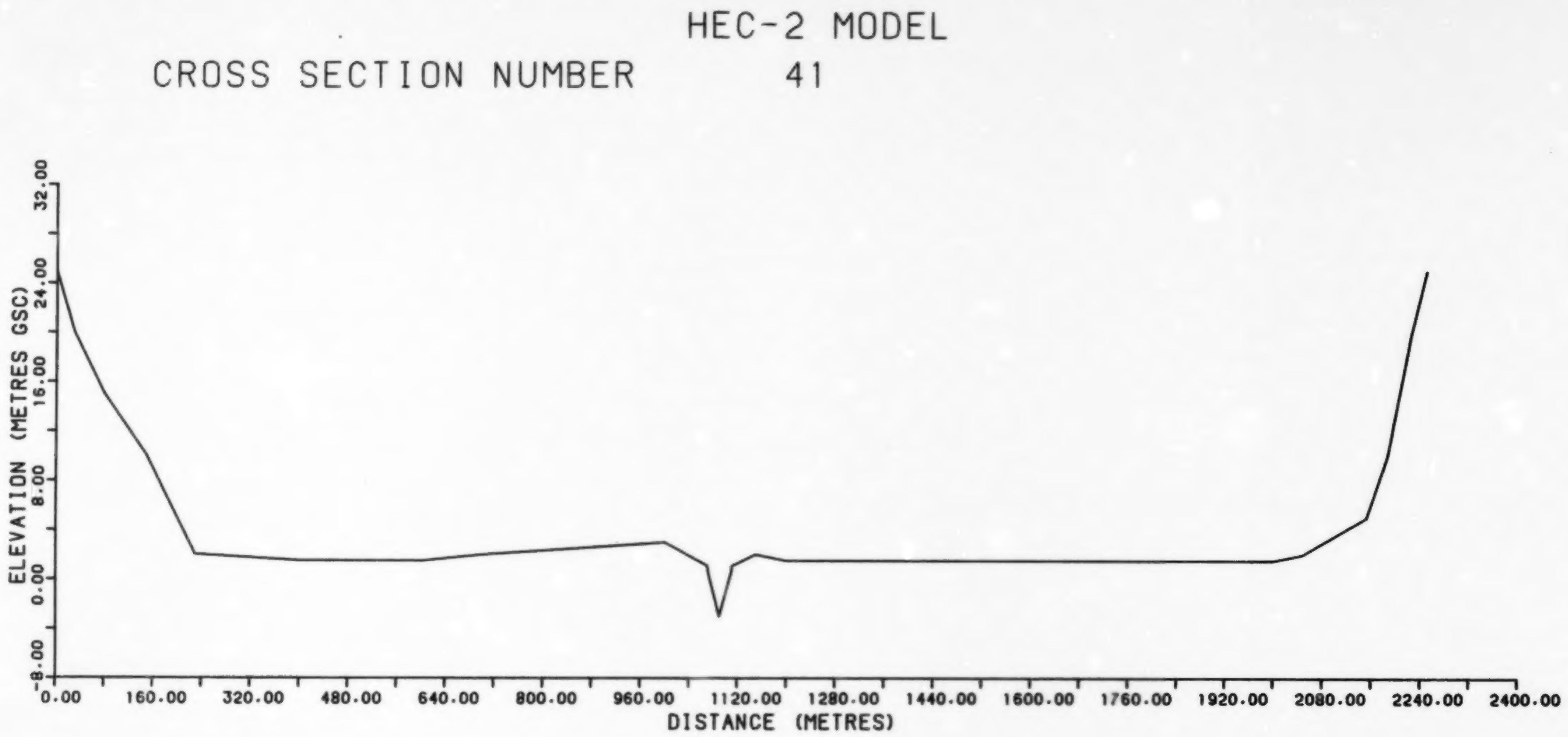

HEC-2 MODEL
CROSS SECTION NUMBER 41
ELEVATION (METRES GSC)
32.00
24.00
16.00
8.00
0.00
-8.00
0.00
160.00
320.00
480.00
640.00
800.00
960.00
1120.00
1280.00
1440.00
1600.00
1760.00
1920.00
2080.00
2240.00
2400.00
DISTANCE (METRES)

FIGURE B-16

HEC-2 MODEL

CROSS SECTION NUMBER 42

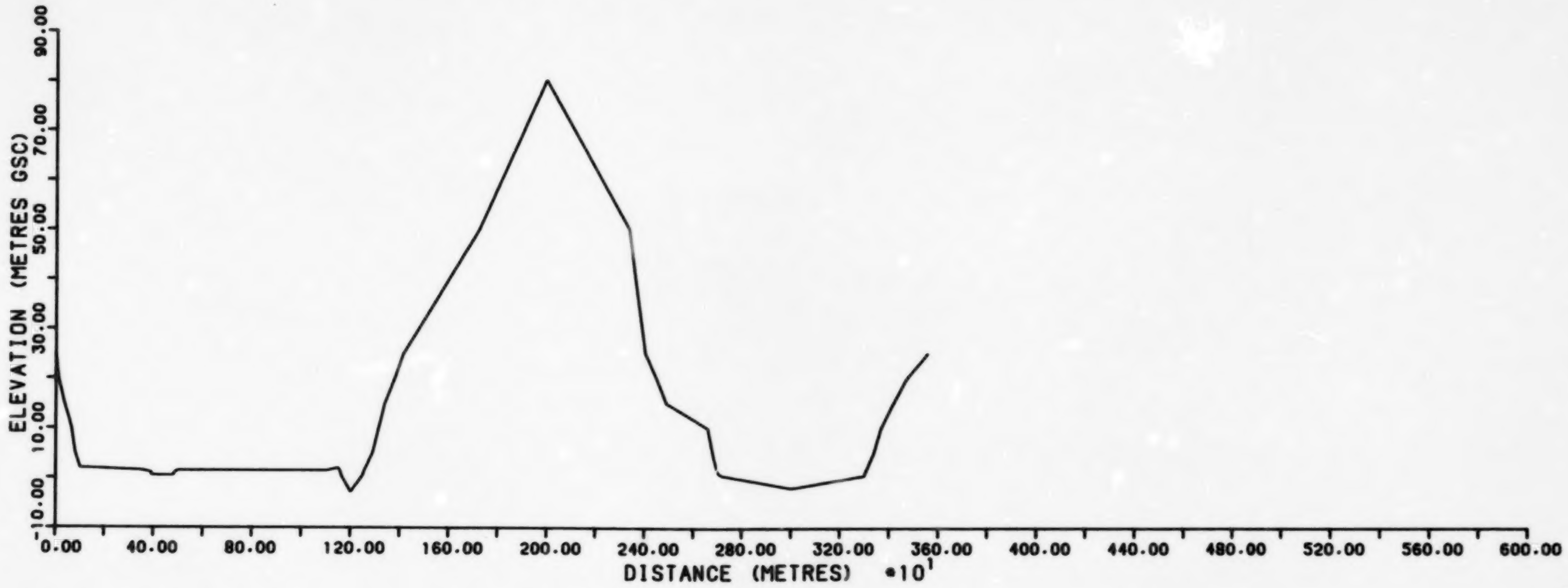


FIGURE B-17

HEC-2 MODEL

CROSS SECTION NUMBER 43

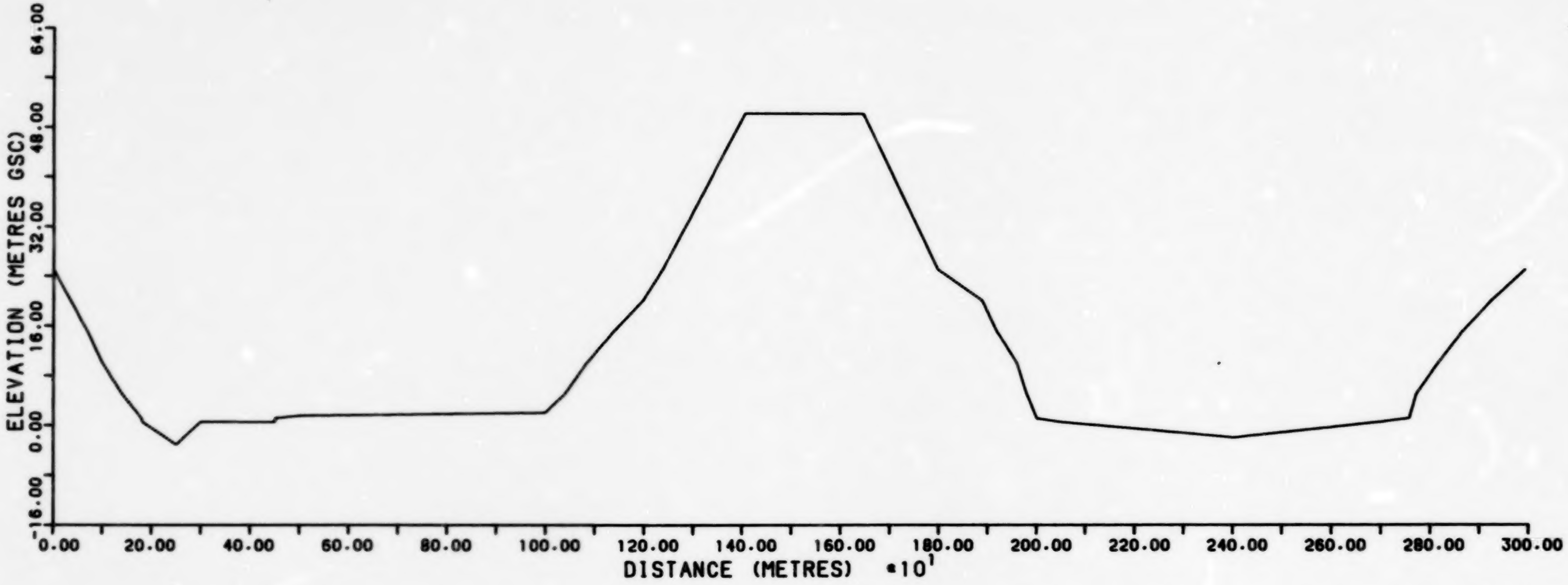


FIGURE B-18

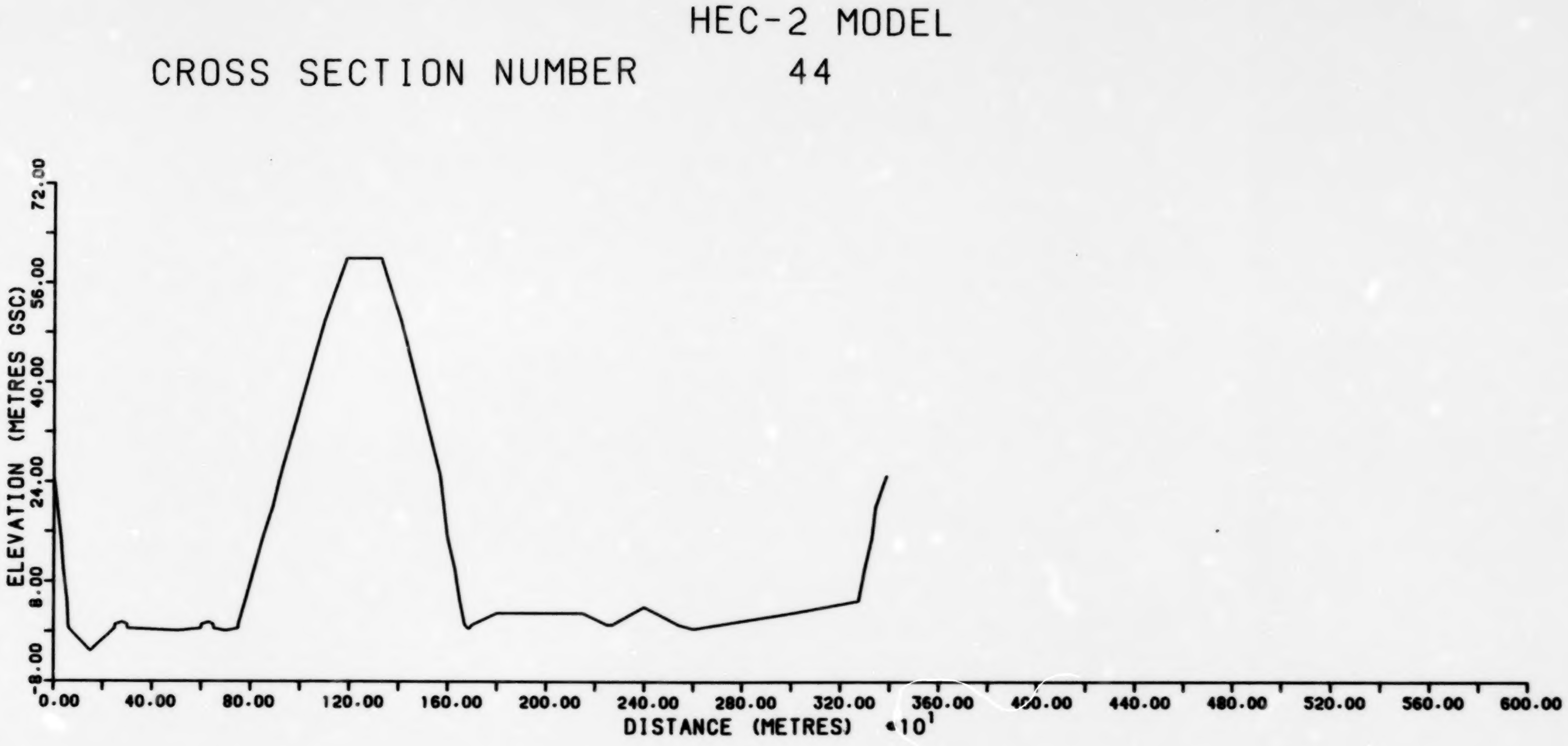

HEC-2 MODEL
CROSS SECTION NUMBER 44
ELEVATION (METRES GSC)
-8.00
8.00
24.00
40.00
56.00
72.00
0.00
40.00
80.00
120.00
160.00
200.00
240.00
280.00
320.00
360.00
400.00
440.00
480.00
520.00
560.00
600.00
DISTANCE (METRES) *10^1

FIGURE B-19

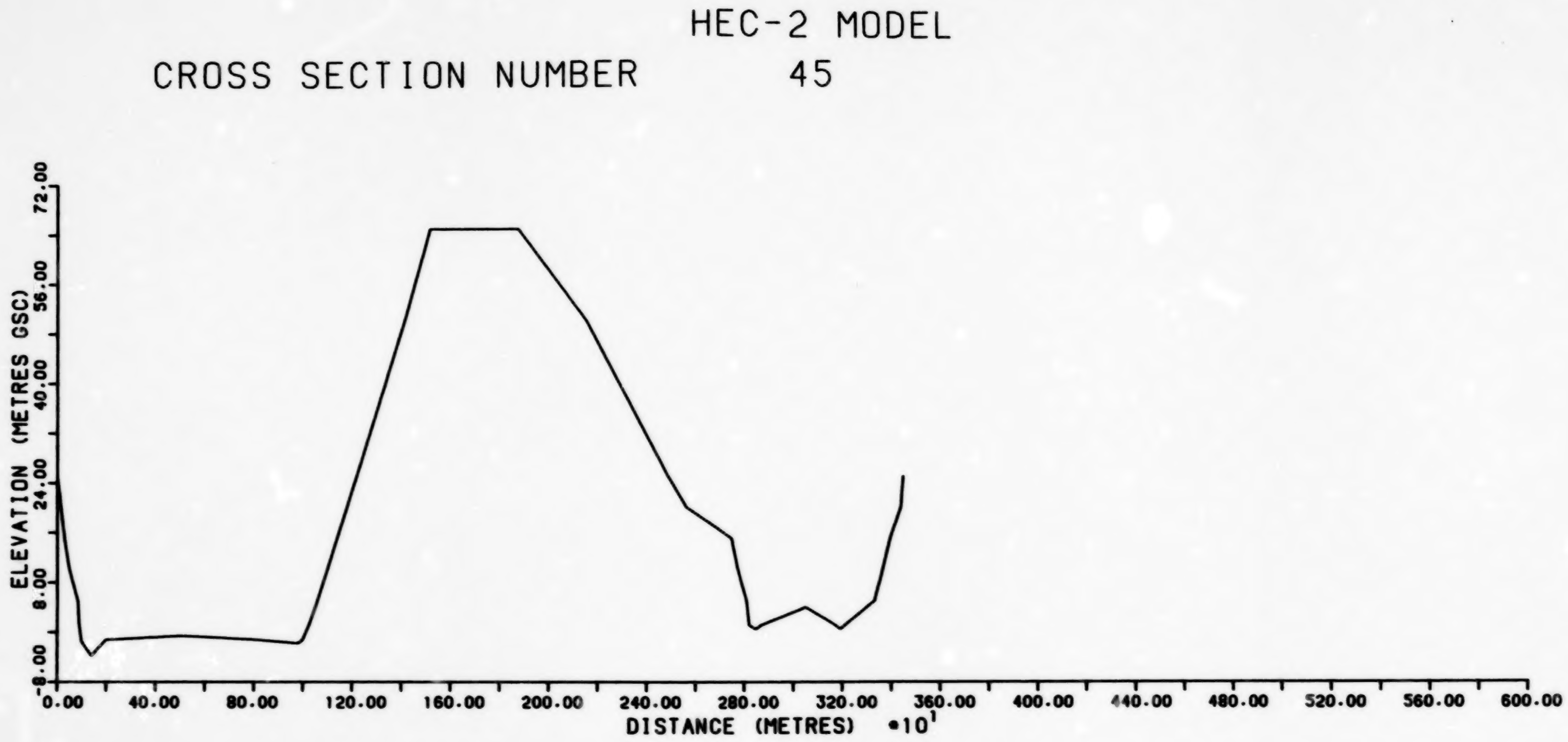

HEC-2 MODEL
CROSS SECTION NUMBER 45
ELEVATION (METRES GSC)
-8.00
8.00
24.00
40.00
56.00
72.00
0.00
40.00
80.00
120.00
160.00
200.00
240.00
280.00
320.00
360.00
400.00
440.00
480.00
520.00
560.00
600.00
DISTANCE (METRES) *10¹

FIGURE B-20

HEC-2 MODEL

CROSS SECTION NUMBER 46

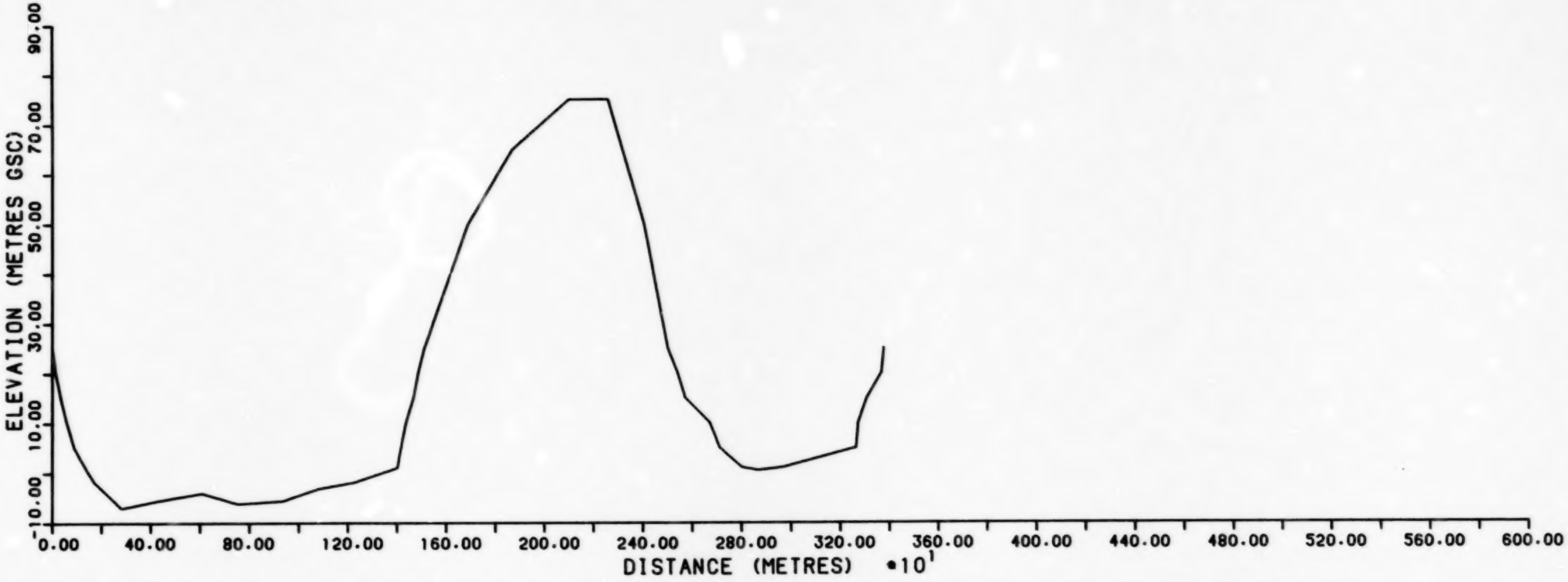


FIGURE B-21

POOR

QUALITY

ORIGINAL